A FRAMEWORK FOR THE EVALUATION OF THE STRUCTURAL SAFETY OF EXISTING CONCRETE GRAVITY DAMS

by

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ABSTRACT

Addressing the aging and deterioration of dams is a relatively new challenge in the dam engineering field in South Africa. The average life expectancy of a dam is approximately 50 years, but they can normally be used for much longer periods than this. Maintenance and rehabilitation are vital to ensure that they achieve their design service lives and for the extension of their service lives. The rehabilitation of dams to extend their service lives and/or to ensure that they comply with modern stability criteria can be an extremely lengthy and costly exercise. It would therefore be valuable to do some research into the process of evaluating the structural safety of dams.

The focus of this research study is specifically on the evaluation of the structural safety of large concrete gravity dams. The purpose of the research is to investigate the most commonly used and accepted methods to evaluate the structural safety of concrete gravity dams and develop a framework that can be used for the evaluation of the structural safety of concrete gravity dams.

For the purpose of this research study, an existing large concrete gravity dam was evaluated as a case study. The dam is approximately 93 years old and provides water to the nearby local municipality for domestic purposes. According to the first and second Dam Safety Evaluation (DSE) reports the dam does not comply with modern stability criteria for concrete gravity dams. The findings of these dam safety evaluation reports led to the dam being labelled as essentially "unsafe" in the case of the occurrence of a large flood. Typically, this would mean that the dam must be rehabilitated to improve its safety. However, some engineers believe that this is not the case and that major rehabilitation is not necessary.

In this research study a framework for the evaluation of the structural safety of existing concrete gravity dams was developed based on lessons learnt from the literature review and the case study. This recommended framework can be useful as a guide for future safety evaluations of existing concrete gravity dams. The potential benefits of using this recommended framework includes avoiding unnecessary rehabilitation work, as well as significant time and cost savings.

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LIST OF SYMBOLS AND ABBREVIATIONS

AAR	Alkali Aggregate Reaction
ASR	Alkali-Silica Reaction
AVM	Ambient Vibration Method
AVT	Ambient Vibration Testing
APP	Approved Professional Person
BIS	Bureau of Indian Standards
DWS	Department of Water and Sanitation
FEM	Finite Element Method
FERC	Federal Energy Regulatory Commission
FSL	Full Supply Level
FSI	Fluid Structure Interaction
g	Gravitational acceleration
ICOLD	International Commission On Large Dams
MCE	Maximum Credible Earthquake
NOC	Non-Overspill Crest
NWA	National Water Act
OBE	Operational Basis Earthquake
PGA	Peak Ground Acceleration
RCC	Roller Compacted Concrete
RDE	Recommended Design Earthquake
SABS	South African Bureau of Standards
SANCOLD	South African National Committee on Large Dams
SEF	Safety Evaluation Flood
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation

CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

Addressing aging and deterioration of dams in South Africa is a relatively new challenge in the field of dam engineering. Most of the large dams (more than 30 m high) in the country were built and are owned by the Department of Water and Sanitation (DWS) in the 20th century. The construction of large DWS owned dams reached a peak in the 1980's (with the construction of large concrete dams peaking in the 1960's and 1970's, see Figure 1). There was a steep decline in the construction of new large dams after this period.

The average life expectancy of a dam is 50 years (Hotchkiss, Barber & Wohl, 2001: IX-64). However, they can typically be utilized for much longer periods than this. Maintenance and rehabilitation are vital to achieve and extend their service lives. There has been a general transition in dam engineering focus in the country from design and construction to maintenance and rehabilitation (Oosthuizen et al., 2010:239).

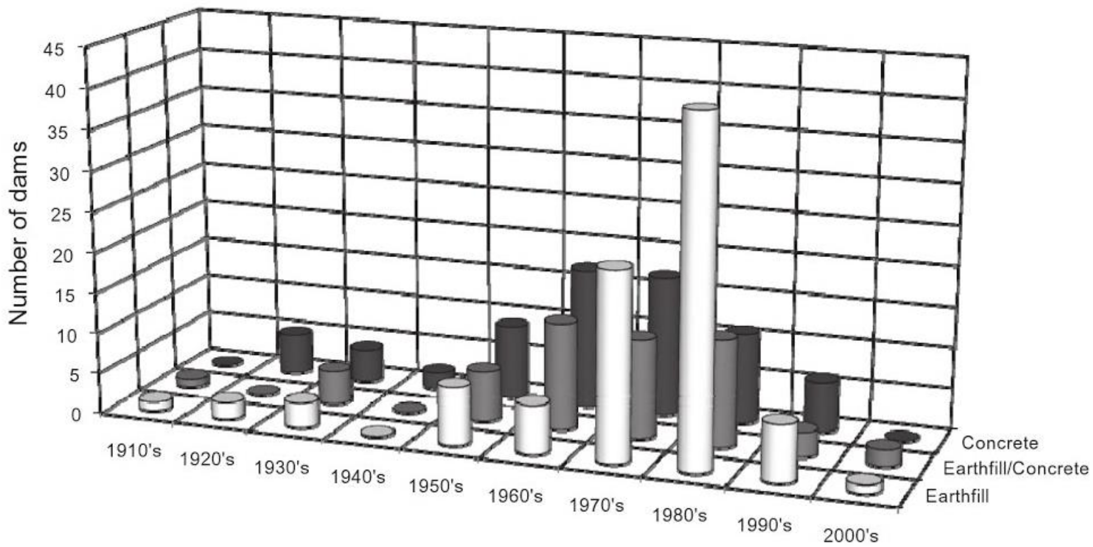

Figure 1: Number and type of large dams constructed and owned by the DWS in the 20th century (Oosthuizen et al., 2010:240)

In January 1987, dam safety legislation was implemented in South Africa with the purpose of improving the safety of new and existing dams with a safety risk to reduce potential harm to the public, damage to property or resource quality (Republic of South Africa, 1998:77). This legislation is currently still in force via the National Water Act (Act 36 of 1998) and the Dam Safety Regulations (published in

Government Notice R. 139 of 24 February 2012). The legislation essentially requires dam safety evaluations (DSE's) to be carried out at regular intervals (generally 5 years) to thoroughly assess the state of dams and reduce the risk of a dam failure. A DSE is basically an evaluation of the overall safety of a dam, and consists of an evaluation of the available information, geology, hydrology, hydraulics, structural safety (or stability), monitoring, an overall risk assessment and a visual inspection of the dam. The results and recommendations of the DSE should then be implemented to maintain and/or improve the safety of the dam.

The are many different types of dams and each have their own benefits and applications as well as methods of being designed and analysed. Henkel (2015:58) reported that the earliest known dam (Jawa Dam) was a gravity dam, built in 3000 BC (approximately 5 000 years ago). The United States Bureau of Reclamation (USBR, 1987:315) describes a concrete gravity dam as a dam which is proportioned in a way that its own weight provides the major resistance to the forces acting upon it. For this dissertation, the focus is on the evaluation of the structural safety of concrete gravity dams.

Various methods have been developed to design and analyse the structural safety of gravity dams. Structural analysis methods can either be of a deterministic or probabilistic nature. For gravity dams, there are two generally accepted deterministic structural design and analysis methods, namely, the "Classical Method" and the finite element method (FEM) analysis.

For more than 100 years the "Classical Method" (also called the "Conventional Method" or "Gravity Method") has been used to design gravity dams (Watermeyer, 2006:2). The USBR (1987:330) regards the method as being substantially correct, except for horizontal planes near the base of the dam where yielding is reflected in stress calculations. In this case the use of the FEM analysis is recommended. Mr Hans Durieux, a Senior Specialist Engineer at the Department of Water and Sanitation (with approximately 44 years of experience in the industry), explained in his master's dissertation (Durieux, 2008) that this method, which is based on Bernoulli's "shallow beam theory", has received a great deal of criticism by academics and experienced engineers - despite its popularity. He explains that this criticism is due to the fact that the method has many limitations (which will be discussed later in Section 2.6.2 of the literature review). Durieux (2008:1), reported that the popularity of the Classical Method is linked to its straight forward approach and the fact that manual (hand) calculations can be done. Furthermore, he emphasised that the method has proved to be a conservative standard which has been used to design and optimise most gravity dams.

With the advancement of technology in recent decades, new methods have been developed to assess the safety of dams. The FEM can be used to optimise the design and analyse gravity dams more efficiently than before. This method is more accurate and cost efficient. It has now become a popular tool for analysing structures.

Durieux (2008:131, 147, 165, 174) compared the two methods (Classical and FEM) in a number of case studies and reported that the classical method is indeed generally a more conservative approach. Durieux (2008:1) also reported that the FEM is a more powerful design tool and enables engineers to make better economic optimizations for dam rehabilitation.

Risk-based, or probabilistic, structural safety evaluations have also become an integral part of both the design and analysis processes. Probabilistic methods account for the limitations of conventional deterministic methods (i.e. the Classical Method and FEM) by making provision for the variations and uncertainties in material properties, loads and analysis criteria. As a result, this method provides useful information for decision makers. Oosthuizen et al. (1991:144), recommended that risk-based evaluations should be regarded as complementary and not contrary to traditional deterministic approaches. In terms of probabilistic analysis methods, the guidelines provided by Oosthuizen (1985) in his PhD dissertation are extensively used by the DWS in the probabilistic evaluation of the structural safety of their dams.

An overall evaluation of the structural safety is typically achieved by using the results of one or the combination of the deterministic and probabilistic structural analyses. Currently, there are no formal gravity dam design or analysis codes, only guidelines exist. Only using the "Classical Method" with conservative assumptions for the structural analysis of existing concrete dams can result in expensive and unnecessary remedial work. Advanced methods exist and should be used before choosing to do remedial work.

In the opinion of the author, the final evaluation should be holistic, considering all site-specific factors and using the most advanced, generally accepted methods of analysis before deciding to rehabilitate a dam. The focus of this dissertation is therefore on the holistic structural safety evaluation of existing concrete gravity dams.

1.2 PROBLEM STATEMENT

A large number of the older DWS owned dams in South Africa (built in the 20th Century) do not comply with present day design criteria (Oosthuizen et al., 2010:239). According to the DWS (2015: iii,9), 78 % of the top 100 dams on the Dam Safety Office's (DSO) priority list of dams with shortcomings belong to the DWS and municipalities (with 52% being DWS owned). The results of the structural analyses using the "conservative" and "simple", classical method may have led to some of these dams to be classified as being structurally unsafe. To rehabilitate all of these dams and make them comply to modern day design "standards" would be an extremely costly and lengthy exercise.

However, with the use of modern structural analysis methods and the consideration of all the available information and that the structure is existing, a more realistic structural safety evaluation can be achieved. In turn, more economical optimizations in terms of rehabilitation can be accomplished and costs of unnecessary, expensive rehabilitation can be avoided.

Therefore, the purpose of this research study is to:

- investigate the most commonly used and accepted methods used to evaluate the structural safety of concrete gravity dams and
- develop a framework used for the structural safety evaluation of concrete gravity dams

The framework developed in this dissertation would provide a useful guide for future evaluations of the structural safety of gravity dams.

1.3 DISSERTATION OUTLINE

In Chapter 2 a literature review is presented. The literature review gives the reader a concise background of gravity dams, the potential failure modes and historical failure incidents, as well as an insight to common structural safety evaluation methods. Chapter 3 presents a case study, namely, the structural safety evaluation of Nqweba Dam. In Chapter 4, a proposed framework for the safety evaluation of concrete gravity dams is discussed. The concluding remarks of this dissertation and recommendations for future research are presented in Chapter 5. The references used in this minor dissertation are presented in Chapter 6.

CHAPTER 2 – LITERATURE REVIEW

2.1 INTRODUCTION

The Department of Water Affairs and Forestry (2001:2) defines a dam as a man-made barrier generally constructed of earthen materials or concrete for the purpose of controlling or storing water for a variety of uses. Dams can be classified according to their use, hydraulic design, structural form, and materials from which they are constructed. For this dissertation, dams will be classified according to their structural form. The most commonly used structural forms are:

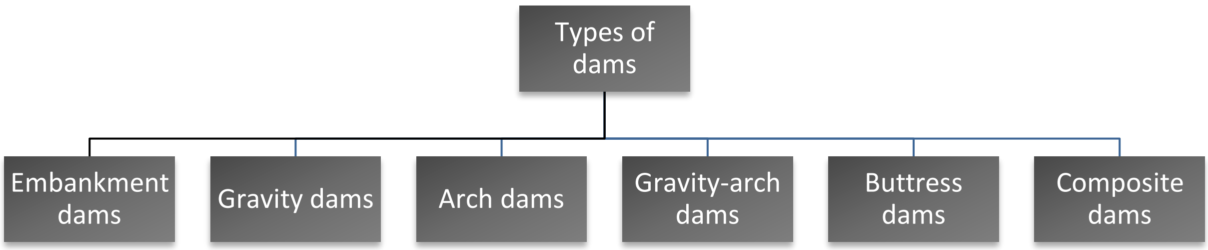

Figure 2: Typical classification of dams according to structural design.

The focus of this dissertation is on the structural safety evaluation of gravity dams. Hence, the literature study first gives the reader a background on the stability mechanism and the various types of gravity dams. It then describes the loads acting on gravity dams and potential failure modes to consider with a brief look at some historical failure incidents. Furthermore, it briefly explores the commonly used methods of evaluating the structural safety of concrete gravity dams. Thereafter, the load combinations and factors of safety are discussed. This chapter is then concluded with a brief chapter closure.

2.2 GRAVITY DAMS

The USBR (1987: 315) describes a concrete gravity dam as a dam which is proportioned in a way that its own weight provides the major resistance to the forces acting upon it. A typical cross section of a gravity dam showing its stability mechanism is presented below in Figure 3. Gravity dams are generally constructed on a straight axis in plan, but may be slightly curved or angled in some cases to meet site specific conditions (United States Army Corps of Engineers [USACE], 1995:2-1). The upstream face of a gravity dam is typically vertical (can be slightly sloped in some cases) and the downstream face, is usually always sloped. Gravity dams are suitable for sites that have a reasonably sound rock foundation - although low structures may be founded on alluvial foundations if adequate cutoffs are provided (USBR, 1987:62).

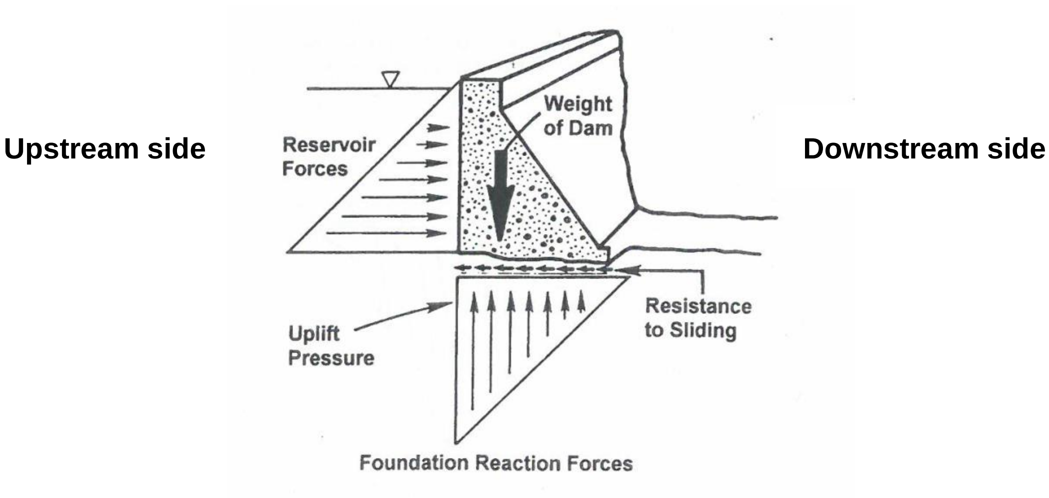

Figure 3: Illustration showing a typical example of the stability mechanism of a gravity dam (Department of Water Affairs and Forestry, 2001:19)

Gravity dams are either constructed using masonry, conventional mass concrete or Roller Compacted Concrete (RCC). In the past (several thousands of years B.C), gravity dams were constructed using uncemented masonry (USBR, 1987:315). As time commenced, masonry dams were constructed using various types of mortar (e.g. cement and concrete) in combination with the masonry blocks. In recent times, however, gravity dams are more commonly constructed by conventional mass concrete or RCC and are referred to as "concrete gravity dams" (International Commission On Large Dams [ICOLD], 2007:21).

2.2.1 Masonry gravity dams

Masonry gravity dams are constructed using brick, stone, rock or concrete blocks sometimes joined with mortar. The Hely-Hutchinson dam (completed in 1904) and Woodhead (completed in 1897) dam on Table Mountain in Cape Town, South Africa are typical examples of masonry gravity dams. Figure 4 displays a downstream view of the Hely-Hutchinson dam.

Figure 4: Hely-Hutchinson dam constructed (completed) in 1904 is a typical example of a typical masonry gravity dam (Crawford, 2013)

2.2.2 Mass concrete gravity dams

As the name suggests, conventional mass concrete gravity dams are constructed using large volumes of concrete. Due to the expansion and shrinkage caused by the heat generated during the cement hydration process of conventional concrete, the size and rate of placement is limited and necessitates building in monoliths to meet crack control requirements (USACE, 1995:2-1). As a result, these dams are vertically constructed in individually stable large blocks (or monoliths), by placing large quantities of concrete (typically in 1.5 to 3 m lifts) into pre-set forms and consolidating the concrete by means of pneumatic vibration. To meet crack control requirements, it is also vital that the concrete mixture is of the correct proportions and that environmental influences (like temperature and moisture) are controlled. An example of a mass concrete dam is presented in Figure 5.

Figure 5: Nqweba dam, constructed (completed) in 1920 is a typcial example of a mass concrete gravity dam. The stepped downstream face is quite a unique finish.

2.2.3 Roller-compacted concrete (RCC) gravity dams

First used in the early 1980's, roller-compacted concrete (RCC) gravity dam construction is a relatively new, efficient and effective alternative method to conventional mass concrete dam construction. The concrete mix used in RCC construction, often referred to as rollcrete, is a relatively "dry" concrete mix consisting of well graded gravel and sand with a low cement and water content. This mix is placed in layers and compacted using heavy mechanical rollers (similar to the method used in embankment dam construction). RCC gravity dam construction is referred to as horizontal construction with 300 mm construction lifts between horizontal joints. As a result, there are more horizontal joints than in mass concrete gravity dams - which typically have 2.4 m lifts.

According to the USBR (1987:315), this method of construction is quicker, requires less labour and is also more cost efficient than constructing conventionally placed mass concrete dams. However, some concerns associated with this method are bond strength and permeability along lift surfaces, cooling requirements and the incorporation of transverse contraction joints (USBR, 1987:315). Figure 6 presents a photo of the raising of Flag Boshielo dam, an existing RCC dam raised using RCC. Most new gravity dams are now constructed using RCC construction.

Figure 6: Example of an RCC dam: Flag Boshielo dam (originally constructed in 1987 and raised from 2004 to 2006).

2.3 LOADING ON GRAVITY DAMS

When designing or analysing gravity dams, it is imperative to know which forces may have an effect on its stability and stresses. The loads acting on gravity dams (depicted in Figure 7) include; dead load, external hydrostatic pressure, internal hydrostatic pressure, earth and silt load, temperature load, alkali-aggregate reaction (AAR), wind load, wave load, ice load and earthquake load. These various loads which may act on a gravity dam are briefly discussed from an analysis perspective in this section.

It should be noted that an exact determination of the details of the loads acting on a gravity dam cannot always be made. Hence, after the consideration of all available information, the engineer should use his judgement and experience to estimate the intensity, direction and location of these loads (Federal Energy Regulatory Commission [FERC], 2002:3-2). The dead load and external hydrostatic pressure can be determined quite accurately. The other loads, namely, the earth and silt pressure, temperature, AAR, internal hydrostatic pressures, ice load, wind and earthquake load are usually predicted based on assumptions of varying reliability.

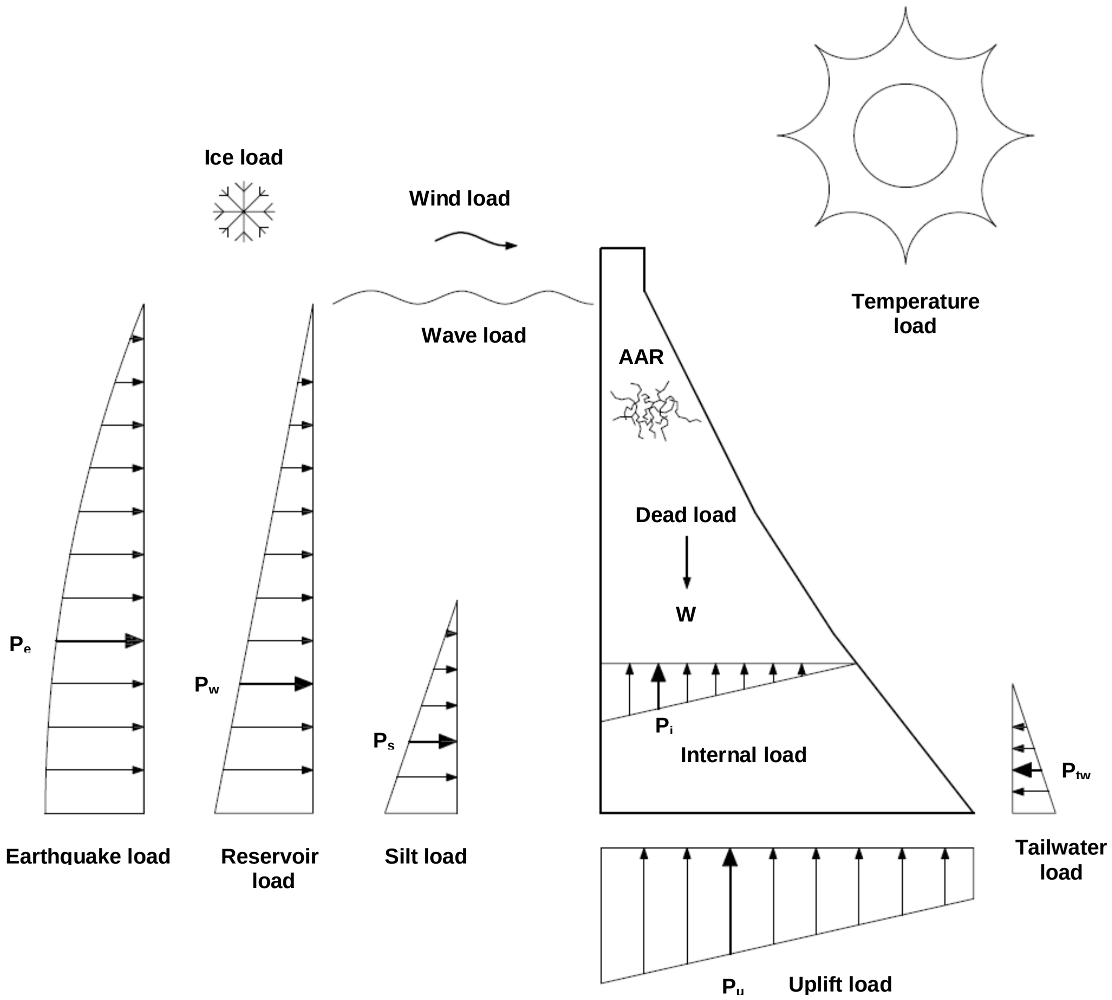

Figure 7: Loads acting on a gravity dam

2.3.1 Dead load

The USBR (1987:321) explains that the total dead load includes the self-weight of the concrete gravity structure as well as the weight of other appurtenant structures such as gates and bridges. The properties of the concrete should preferably be determined by means of a material investigation (to get actual tested values). If this is not possible to attain, then assumed properties can be used. According to FERC (2002:3-2), USBR (1987:316) and US Corps of Engineers (1995:3-3), unless testing indicates otherwise, a reasonable assumption of 24 kN/m^3 for the unit weight of the concrete (γ_c) can be used when calculating its dead load. The force of the weight of the dam wall per unit length (W, in kN/m) is equal to the product of the cross-sectional (A) and the unit weight (γ_c) of the concrete. This force acts vertically downwards at the centroid of the cross-section.

$$W = \gamma_c A \tag{2.1}$$

The weight of small appurtenant structures (in comparison to the dam size) is often regarded as negligible and relatively small voids (such as galleries) within the dam are usually not subtracted from the total dead load. However, the engineer should use his engineering judgement when deciding whether or not to subtract voids or exclude the weight of appurtenant structures.

2.3.2 External hydrostatic pressure

External hydrostatic pressure generally refers to the reservoir and tailwater pressures. A linear distribution of the static water pressure, which increases with depth, should be used. The unit weight of water (γ_w) is normally assumed to be 10 kN/m^3. The external hydrostatic pressure diagram is triangular and has a pressure intensity of $\gamma_w h$ at the base (where h is the depth of water). The resultant force per unit length (P_w in kN/m) of the external hydrostatic pressure is calculated using the following formula:

$$P_w = \frac{1}{2}\gamma_w h^2 \tag{2.2}$$

The force acts at a height of h/3. In overflow sections, at low discharges, the load of the water overflowing the dam is usually considered to be negligible. However, under high flood conditions the engineer should use his judgement as to whether or not to include this load.

2.3.3 Internal hydrostatic pressure

Internal hydrostatic pressure refers to pressures which develop in interstitial spaces such as pores, cracks, joints and seams within the dam wall, at the interface between the dam and its foundation, and within the foundation. These pressures are caused by external hydrostatic forces (i.e. reservoir and tailwater pressures).

According to the USBR (1977:24), the effect of internal hydrostatic pressure reduces the vertical compressive stresses in the concrete on a horizontal section through the dam or at its base and is referred to as "Uplift ".

The distribution of uplift pressure at the dam-foundation interface and within the foundation depends on the presence, depth and spacing of drainage holes, the grout curtain, rock porosity, jointing, faulting, and any other features (such as aprons and high silt levels) that may possibly modify the seepage or flow of water. In the absence of drainage holes or a more detailed analysis, the distribution of the uplift pressure varies linearly from full reservoir pressure at the upstream face to zero or tailwater pressure at the downstream face (USBR, 1987:320). The uplift pressure at the dam-foundation interface and within the foundation is typically reduced by a line of drainage holes drilled from the floor of the foundation gallery into the foundation. The uplift pressure at the line of the drains is usually assumed to be reduced to the tailwater pressure plus one third of the difference between the reservoir and tailwater pressure - the pressure gradient then extends linearly to the headwater and tailwater levels. According to FERC (2002:3-4) uplift reduction can only be assured by the implementation of a comprehensive instrumentation and monitoring system. For all of these cases the pore pressures are usually assumed to act over 100% of the area of the failure plain.

It is a generally accepted practice to assume that pore pressures act over 100% of the area of any section through the concrete. This assumption has been confirmed (for practical purposes) by laboratory tests (USBR, 1977:24). According to the USACE (1995:3-5), in conventional concrete dams, within the body of the dam uplift may be assumed to vary linearly from 50% of the maximum headwater pressure on the upstream face to 50% of tailwater pressure or zero (whatsoever the case may be) at the downstream face. The USACE (1995:3-5) also explained that this simplification is based on the relative impermeability of the intact concrete which prevents the build-up of internal pore pressures. However, they state that cracking at the upstream face or weak horizontal joints may affect this assumption. As many sources (including FERC (2002:3-3) and USACE (1995:3-5)) recommend, in this case as uplift pressures should be calculated as it would at the dam-foundation interface.

It should be noted that the pore pressures may vary over time and are dependent on the boundary conditions and permeability of the material (USACE, 1995:3-4). Uplift pressures must be included in the structural analysis to ensure structural adequacy. These pressures are usually assumed to be unchanged by earthquake loads.

2.3.4 Earth and silt pressure

Earth and silt pressure can be applied in a similar manner to which external hydrostatic loads are applied - namely, a linear distribution along its height increasing with depth. The unit weight of the silt/earth (γ_s) and hydrostatic pressure is normally assumed to be 13.62 kN/m^3 in the horizontal direction and 19.22 kN/m^3 in the vertical direction - that is, in the absence of actual tested values. The resultant force per unit length (P_s in kN/m) of the earth/silt pressure is calculated using the following formula:

$$P_s = \frac{1}{2}\gamma_s h^2 \tag{2.3}$$

The force acts at a height of h/3. In cases where passive and active soil pressures are likely to develop, Rankine's theory of lateral earth pressure should be applied. Rankine’s theory of lateral earth pressure considers the state of stress in a mass of soil when the condition of plastic equilibrium is reached, that is, when shear failure is on the point of occurring throughout the mass (Craig, 2004:162). The state of plastic equilibrium is reached when irreversible strain (failure) takes place at a constant stress. If the wall is rigid and does not move with the pressure exerted on the wall, the soil behind the wall will be in a state of elastic equilibrium.

When using Rankine's theory, the following assumptions are made:

- There is no friction between soil and wall
- Soil is homogeneous and isotropic
- The ground failure surfaces are straight planes

According to Rankine's theory of lateral earth pressure, the pressure exerted by a mass of soil on a retaining structure is either active or passive. Figure 8 shows the active and passive earth pressures exerted on a retaining structure.

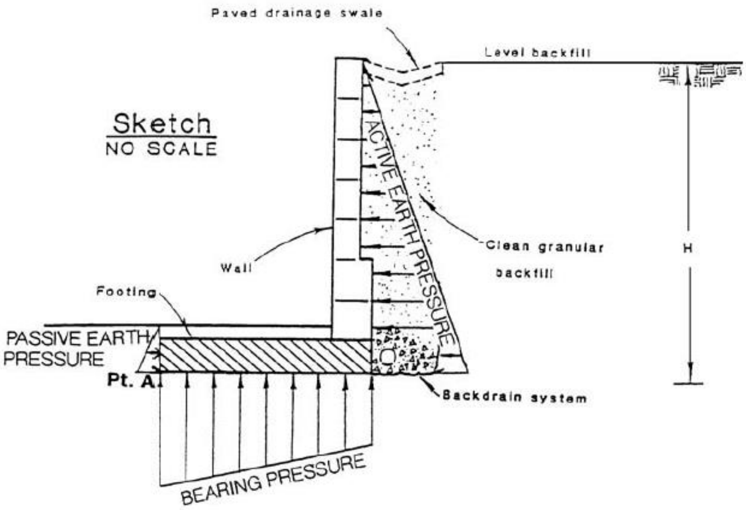

Figure 8: Active and Passive earth pressures acting on a retaining wall (Day, 2010:11.3)

A mass of soil is in an *Active Rankine State* when the wall moves outwards and away from the soil. The lateral earth pressure starts to be reduced until it reaches its minimum value (P_a), where:

$$P_a = K_a \sigma_v' - 2c'\sqrt{K_a} \tag{2.4}$$

$$K_a = \frac{(1 - \sin\varphi)}{(1 + \sin\varphi)} \tag{2.5}$$

And,

P_a	= active resultant force (kN/m)
K_a	= active earth pressure coefficient (dimensionless)
c'	= effective cohesion
φ	= friction angle of the soil (degrees)
σ'_v	= vertical effective pressure (kN/m)

Passive Rankine State

When the wall moves toward the soil, the soil will be in a *Passive Rankine State*. The passive pressure will increase until it reaches its maximum value (P_p), where:

$$P_p = K_p \sigma'_v + 2c'\sqrt{K_p} \tag{2.6}$$

$$K_p = \frac{(1 + \sin\varphi)}{(1 - \sin\varphi)} \tag{2.7}$$

And,

P_p = passive resultant force (kN/m)
K_p = passive earth pressure coefficient (dimensionless)
c' = effective cohesion
φ = friction angle of the soil
σ'_v = vertical effective pressure (kN/m)

2.3.5 Temperature

The volumetric changes caused by thermal expansion (or contraction) transfers stresses across grouted contraction joints into the abutments of the structure. Thermal expansion will cause the dam to wedge itself into the abutment and create a stabilizing effect. Thermal contraction, on the other hand, will have an opposite effect. When doing a 3D analysis of a structure these forces may be significant. In the case of ungrouted contraction joints it is assumed that no transfer of forces occurs, provided that the mean concrete temperature remains below the closure temperature of the contraction joints.

However, the beneficial effect of the stresses caused by temperature expansion is difficult to quantify as it depends on the modulus of deformation of the abutments, which is highly variable (FERC, 2002:3-14).

2.3.6 AAR

Alkali-aggregate reaction (AAR), more commonly referred to as alkali-silica reaction (ASR), is a reaction which occurs when a cement with a relatively high content of alkali hydroxide is used in combination with aggregates containing potentially reactive forms of silica. The alkalis of the cement react with the silica of the aggregate and an alkali-silica gel is formed around the aggregates. This silica gel then swells as it absorbs water from the surrounding environment, giving rise to expansive stresses within the concrete. The expansive stresses caused by the expansion of the silica gel can in turn damage the concrete by causing cracks to form and deterioration of its engineering properties.

The volumetric changes (expansion) due to ASR also affects the cross-valley stresses in concrete gravity dams. These stresses are important when doing a 3D analysis of the structure.

Just like the case for thermal expansion, the quantification of the stabilizing stresses caused by ASR expansion is difficult as it is also dependent on the modulus of deformation of the abutments, which (as mentioned previously) are highly variable (FERC, 2002:3-14). For this reason, the beneficial effects of ASR are typically acknowledged, but not included in calculations.

2.3.7 Wind pressure

According to the Bureau of Indian Standards (BIS, 1998:11), wind pressure exists, but is seldom a significant factor in the design of gravity dams. Hence, wind loads may be ignored in the structural analyses. Other sources also generally omit wind loads.

2.3.8 Wave pressure

Waves formed on the surface of the reservoir by blowing winds may cause a horizontal force on the upper portion of the dam wall. The BIS (1998:11), explains that the force and dimensions of the waves depend mainly on the extent and configuration of the water surface (the fetch), the velocity of the wind and the depth of the reservoir. Furthermore, the BIS (1998:11) and Ali et al. (2012:19) recommend that the total wave force (P_{wave} in kN) can be estimated by the following equation:

$$P_{wave} = 20h_{wave}^2 \tag{2.8}$$

Where, h_{wave} is the height of the wave and the centre of application is at $0.375h_{wave}$ above the water level of the reservoir.

The force caused by waves may be of little significance in terms of the stability of large dams. Wave pressures may be of more importance in their effect on appurtenant structures and freeboard requirements. Once again, the engineer should use his/her judgement (based on the available information) as to whether or not to include this load in stability calculations.

2.3.9 Ice load

Ice that may form on the surface of the reservoir in cold environments may melt and expand, causing an external load on the dam wall. The ice pressure is created by the thermal expansion of the ice (which depends on the temperature rise, thickness, coefficient of thermal expansion, elastic modulus and strength of the ice) and wind drag (which depends on the size and shape of the exposed area, the roughness of the surface, and the direction and velocity of the wind).

Climatology studies usually determine whether or not consideration of ice pressure is suitable. Most dams in the world do not experience ice loading. The engineer should use the climatology study results and his/her judgement to decide whether or not it should be included in the structural analysis.

According to the USBR (1977:11 & 1987:321), the method of Monfore and Taylor (1948) can be used to analyze expected ice pressures if the required basic data is available. In the absence of available data, the USBR (1977:11 & 1987:321-322) recommends that an ice load of 146 (kN/m) acting on the upstream face (area of contact) for an assumed depth of 0.6 m or more can be used as an acceptable estimate to compute pressures.

2.3.10 Seismic/Earthquake load

Earthquakes produce shock waves in every possible direction (x, y, and z directions), which shake the ground upon which a gravity dam structure is built. As a result of these waves, the effect of an earthquake is therefore equivalent to imparting ground accelerations to the foundation of the structure. It is common practice to express these ground accelerations as a percentage of gravitational acceleration (g) i.e. 0.1g. It is also common practise to perform the analysis for only the most unfavourable direction (the horizontal, downstream-upstream direction). A detailed seismic study of the area in which the dam is situated should preferably be performed to determine the Peak Ground Accelerations (PGA's) to be used in the various load cases in the analysis of the dam. In the absence of a detailed seismicity assessment, general seismic hazard maps for the dam site may be used as a guide. In South Africa, the seismic hazard maps provided by Brandt (2001) and Kijko et al. (2003) can be used as a guide for the determination of seismic loading for various locations within the country.

From a design point of view, the following three earthquake loading and response conditions are usually considered; Operating Basis Earthquake (OBE), Design Basis Earthquake (DBE) or Recommended Design Earthquake (RDE), Maximum Credible Earthquake (MCE). The site-specific seismic study would recommend PGA values to use for each condition. From an analysis perspective, however, only the OBE and MCE cases are usually evaluated. According to FERC (2002:3-13), seismic loading does not have to be considered for structures for which the MCE produces a PGA of less than 0.1g at its site.

There are a number of ways in which earthquake loading is applied to the structure. A brief description of its how these loads are applied, from perspective of the more popular analysis methods, is given in Section 2.6.

According to ICOLD (2001), historically, earthquakes have damaged only a few dams significantly. Only six embankment or concrete dams of significant size have been severely damaged by earthquakes. Furthermore, they mention that no concrete dam is known to have failed as a result of an earthquake.

2.4 FUNDAMENTAL POTENTIAL FAILURE MODES

There are two broadly accepted and agreed upon main modes of failure of gravity dams, namely; Overturning (or rotation) and sliding (or shear failure). Ali et al. (2012:22) separates the modes of failure into overturning, compressive, tension and sliding failure. For the purpose of this dissertation the tensile and compressive stress failure will be grouped under material failure. Figure 9 shows the simplified potential failure modes for gravity dams.

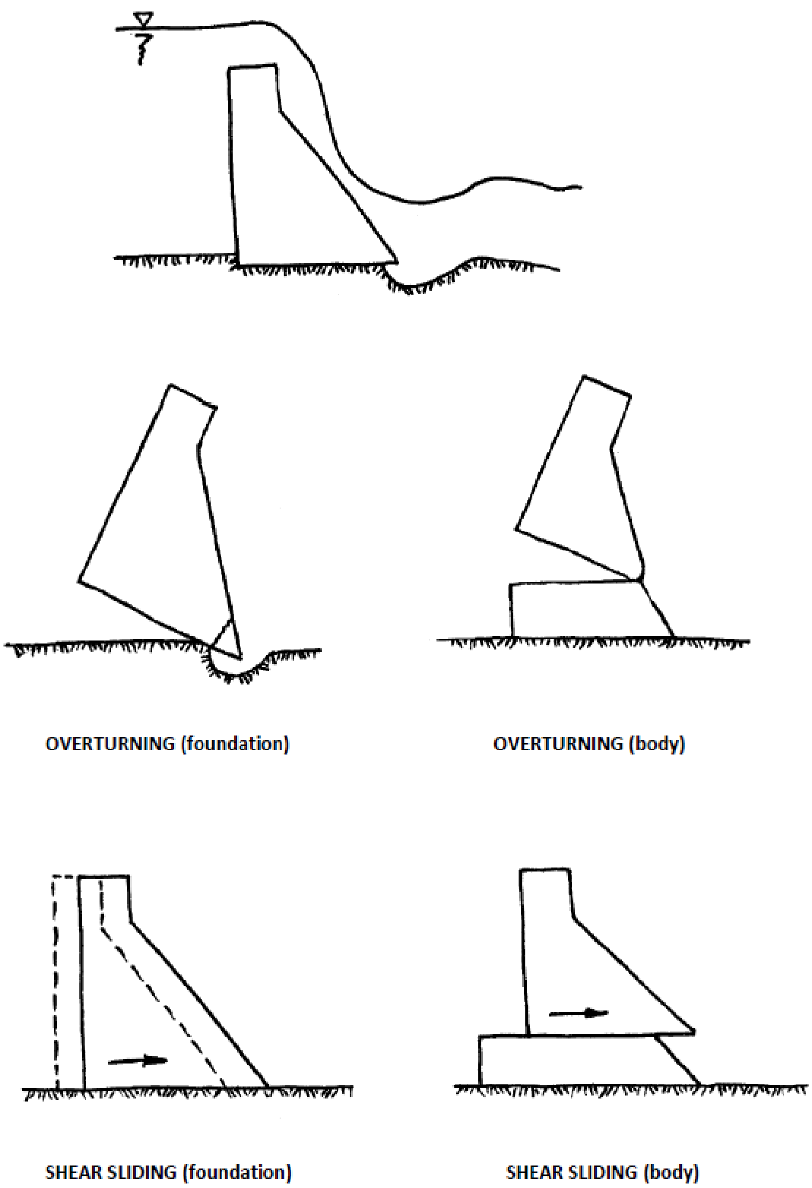

Figure 9: Simplified failure modes of gravity dams (reproduced from Oosthuizen (1985:III-97))

2.4.1 Overturning failure

There are three locations where overturning failure can occur, namely, about the downstream edge of a horizontal joint within the dam body, about the toe/base of the entire dam wall structure, or about a point in a joint below the dam wall in the foundation.

The factor of safety against overturning (FS_O) is defined as the ratio of the sum of the resisting moments (M_R) to the overturning moments (M_O) about the toe.

$$FS_o = \frac{\Sigma M_R}{\Sigma M_o} \tag{2.9}$$

2.4.2 Sliding failure

Sliding failure can also occur in three different places, namely, along a horizontal joint within the wall, the toe/base, or along a joint below the dam in the foundation. The sliding stability safety factor provides a measure of the safety against sliding or shearing on the dam section. The factor of safety against sliding (FS_S) is calculated by using the following formula:

$$FS_s = \frac{cA + V \tan \varphi}{H} \tag{2.10}$$

Where:

c	= Unit cohesion (kN/m^2)
A	= Area of the section considered (length of base in meters)
φ	= coefficient of internal friction (degrees)
H	= Sum of the horizontal forces (kN/m)
V	= Sum of the vertical forces (including uplift forces) (kN/m)

2.4.3 Material failure

Overturning and sliding failure can be triggered by the failure of the materials of the dam or foundation. Material failure can be divided in to tensile and compressive (or crushing) failure. Hence, the stresses within the dam wall and foundation should be evaluated to ensure that the material strengths are not exceeded in both cases. Methods of determining these stresses are discussed in Section 2.6.

It should be noted that in this section (potential failure modes), the modes of failure have been simplified. Only the primary failure mechanisms have been discussed. The actions leading to failure are more complex and fall outside the scope of this dissertation.

2.5 HISTORICAL FAILURE INCIDENTS

2.5.1 General

In this section, some significant concrete dam failure incidents will be discussed, namely, Austin Dam (1911), St. Francis Dam (1928), and Koyna Dam (1967). There some useful lessons to be learnt from each of these incidents.

2.5.2 Austin Dam (1911)

Austin dam was a concrete gravity dam, constructed rapidly by the Bayless Pulp and Paper Company from May to December 1909, to supply water for their company operations. The dam was 15.24 m high and 164.6 m long, with a design capacity of 0.757 million m^3 (Wise, 2005:8). The cost of construction was approximately US $86 million. The dam was situated in the state of Pennsylvania, USA. At the time Austin dam was the largest concrete dam in Pennsylvania.

On the afternoon of Saturday 30 September 1911, the dam failed under static loading with a full reservoir. 78 people were killed, and the incident caused US $14 million worth of property damage. Engineering experts believed that the dam failed as a result of a "sliding failure" due to the weakened foundation rock and uplift pressures. Rose (2013b:1) explains that seepage through the foundation, cracking of the cyclopean concrete (which had large rocks in its matrix), and an inadequate shear key all contributed to the dam's failure. Furthermore, it is known that the dam was plagued with poor design and construction which was influenced by the owner pressurising the designer to keep costs low. The failure of Austin dam led to the first dam safety laws being passed in 1913 to avoid such catastrophes.

Figure 10 shows a plan view illustration of the Austin Dam failure. Figure 11 shows a photograph of the remains of Austin Dam.

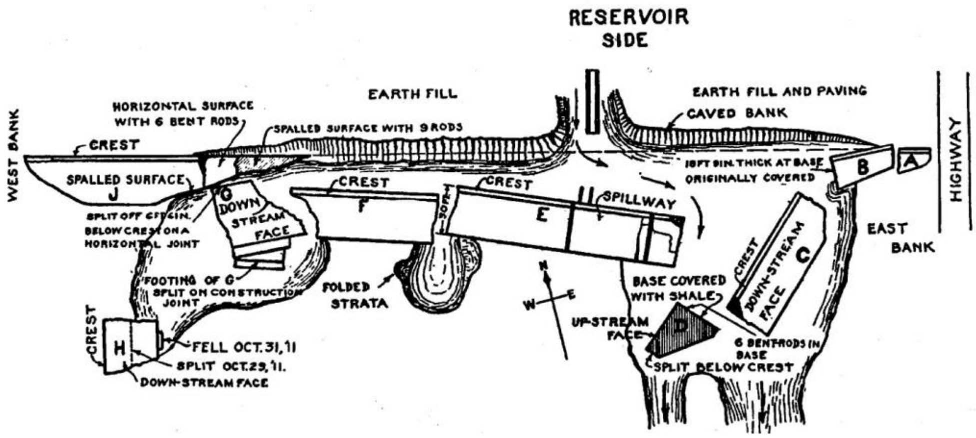

Figure 10: Plan view of the Austin Dam after failure (McKibben, 1912)

Figure 11: The remains of Austin Dam (Rose, 2013a)

2.5.3 St Francis Dam (1928)

St Francis dam was a 62.4 m high gravity-arch dam, which was constructed between 1924 and 1926. The dam was located in the San Francisquito Canyon (in the Los Angeles County, Southern California, USA) and was built by the Los Angeles Bureau of Water Works and Supply to be used as a backup water supply for Los Angeles in case the flow of Owens Valley water was interrupted (USBR, 1998:70). The dam had a capacity of 46.9 million m^3 and was approximately 210 m long.

The dam was knowingly constructed on a fault between red conglomerate (on the right abutment) and mica schist (on the canyon floor and left abutment). The left abutment of the dam, however, was unknowingly constructed on a large, but old paleo-landslide (Rogers, 2006). It should be noted that the design calculations (and geology) were not reviewed externally. The chief engineer who sited the dam (William Mulholland), stated that he was unable to secure enough funds to hire a qualified geologist.

St Francis Dam failed close to midnight on 12 March 1928 - two years after its construction was completed. 432 people were killed. It was recorded as being one of the worst American civil engineering disasters of the 20th Century.

The dam failed under normal hydrostatic load, with no unusual weather conditions and no known seismic activity (USBR, 1998:70). Many investigations concluded that the dam failed as a result of defective or inadequate foundations and the lack of defence measures. According to the USBR

(1998:72), a commission appointed by the Governor concluded that the dam failed due to water percolation and erosion near the fault zone, followed by flow towards the left abutment causing erosion and landslides on that side. Furthermore, they report that another panel appointed by the Santa Clara Valley ranchers concluded that failure was initiated by sliding of an ancient landslide on the left abutment.

Figure 12 presents an illustration of the initial failure mechanism. Figure 13 presents a photograph of the remnants of St. Francis Dam. The foundation was only drained at the remaining middle section seen in the photograph. The undrained left and right abutments failed. According to Rogers (2006:72-73) the St. Francis Dam had several (more than 10) design deficiencies. Furthermore, failure of the dam cannot be linked to one specific shortcoming - seven modes of failure were analysed, and the imminent failure was predicted on each occasion.

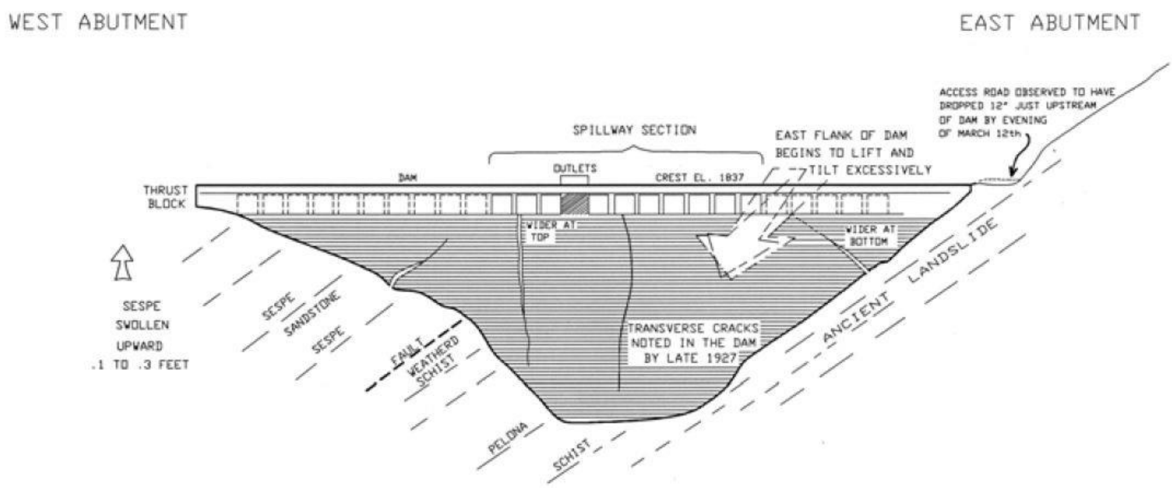

Figure 12: Illustration of an elevation view of the dam with transverse cracks on the downstream face which occurred a year after construction (Rogers, 2006:64)

The failure of St. Francis Dam brought about much appreciation for the inclusion of sound engineering geologic input, the inclusion of uplift forces in siting, design and evaluation of dams, and also the external review of design calculations.

Figure 13: The remnants of St. Francis Dam - only the sloping abutments failed. Photo from the C.H. Lee Collection, U.C Water Resources Centre Archives, Colourized by Pony Horton (Rogers, 2006:34).

2.5.4 Koyna Dam (1967)

Koyna Dam is a 103 m high concrete gravity dam, situated on the Koyna River in the south-western region of India. Its wall is 853 m long and the dam has a capacity of 310 million m^3. Construction of the dam began in 1954 and ended in 1963. The dam was constructed to abate monsoon flood waters, provide irrigation water to the east and provide power to the west (USBR, 1998:40). The dam was constructed primarily from rubble concrete.

On 11 December 1967, the dam was subject to the seismic loading caused by the Koyna Earthquake (6.5 on the Richter Scale) which subjected the dam to ground accelerations that were almost 10 times the designed value (0.05g). During the earthquake, the reservoir was at 11 m below FSL. According to the USBR (1998:40), until the occurrence of the earthquake, the area in which the dam is situated was regarded as stable and non-seismic. Small ground motions were, however, noticed during filling of the dam.

The dam survived the earthquake without major loss of water (USBR, 1998:40). However, the dam suffered significantly deep horizontal cracks on both the upstream and downstream sides in areas where stress concentrations were expected to develop (i.e. in the upper part of the wall, approximately at the level of the change in downstream slope).

The cracks were grouted, and the dam was strengthened by vertically pre-stressed cables in the upper part of the non-overflow sections and adding concrete buttresses to the downstream face. The spillway section was also later in 2006. The dam still stands today and is expected to be able to withstand future earthquakes of higher magnitudes than that of the Koyna Earthquake.

Figure 14: Koyna dam (Parikh, 2011)

2.6 STRUCTURAL SAFETY EVALUATION METHODS

2.6.1 General

In this section the most commonly used and accepted methods of analysis will be discussed. The Classical Method is a simple, conservative method generally considered to be sufficient for the analysis of most gravity dams. The more advanced FEM analysis may be required for more complex cases.

2.6.2 The Classical Method

The "Classical Method" (also called the "Conventional Method" or "Gravity Method") was the most popular method used to design and analyse the stability of gravity dams. The method follows a simple approach which can be done by hand calculations - without the use of a computer. The classical method is relatively cheap and can be used for the design and analysis of small dams - where design costs are critical.

According to Leclerc, Léger and Tinawi (2001:14), the evaluation of the structural stability of the dam against sliding, overturning and uplifting is performed by a stress analysis to determine the crack length and compressive stresses on a failure plane considered and a stability analysis to determine safety margins against sliding and position of the resultant of all forces acting on the plane. The analysis is normally done for each horizontal construction joint (the potential failure planes) within the dam wall.

Ali et al. (2012:23) explains that the assumptions made in this method are:

- the dam is considered to be composed of a number of 1m thick cantilevers, which act independently of one another
- no loads are transferred to the abutments by beam action
- the foundation and dam behave as a single unit
- the material in the dam body and foundation are isotropic and homogeneous, and
- the stresses developed within the dam and foundation are within elastic limits and no movement in the foundation is caused by the transfer of loads

The Euler-Bernoulli shallow beam theory is used to calculate the stress distribution along a potential horizontal failure plane (See equations below). The stresses exerted along a horizontal plane within the dam or foundation are usually calculated using the following formulae:

$$x = \frac{\Sigma M}{\Sigma V} \tag{2.11}$$

$$e = \frac{b}{2} - x \tag{2.12}$$

$$\sigma_{toe} = \frac{\Sigma V}{b}(1 + \frac{6e}{b}) \tag{2.13}$$

$$\sigma_{heel} = \frac{\Sigma V}{b}(1 - \frac{6e}{b}) \tag{2.14}$$

Where:

x	= distance of the resultant force R from the toe (metres)
e	= eccentricity of resultant force from the centre of the base (metres)
b	= Uncracked length of base (metres)
σ_{toe}	= normal stress at toe (kPa)
σ_{heel}	= normal stress at heel (kPa)

When e > b/6, the normal stress at the heel will be negative or tensile. Figure 15 shows the eccentricity (e) and distance (x) of the resultant force from the toe of the dam wall.

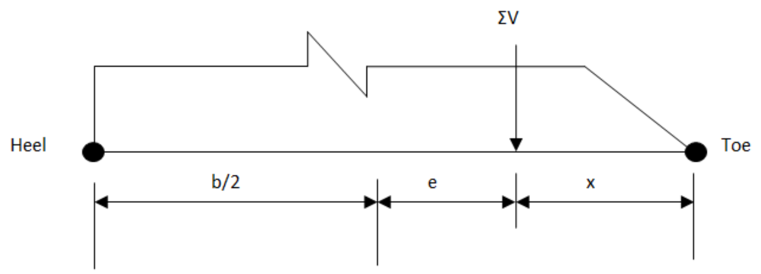

Figure 15: Eccentricity of resultant force

Rigid body equilibrium is used to determine the internal forces acting on this plane. Forces acting on the wall can be calculated as discussed in Section 2.3. The overturning safety factor is calculated as explained in Section 2.4.1. Sliding stability is calculated using Coulomb's friction equation (See Section 2.4.2).

Furthermore, a crack analysis should be done to determine the crack length, starting at the upstream side propagating in the downstream direction. Theoretically speaking, cracking occurs when the tensile strength of the concrete is exceeded. In a cracked analysis, however, a crack is generally assumed to form if the vertical force is tensile. The crack length should be calculated as it would influence the factor of safety against sliding as well as the correct uplift pressures. The crack analysis can also be done at each horizontal construction joint along the height of the dam. Ebeling et al. (2000) compares the methods used by three federal agencies to calculate the cracked lengths and can be used as a reference to calculate cracked length. Durieux (2015a:243) describes the simplified method of calculating the crack length under a gravity wall as developed by Jansen (1988). Leclerc, Léger and Tinawi (2001) also discusses the crack analysis in detail. It should, however, be noted that these methods are approximate, sensitive, and conservative approaches. The non-linear FEM analysis is a more reliable method for determining the crack length (Durieux, 2015a:244).

Some limitations of the Classical Method include:

- the fact that the Euler-Bernoulli shallow beam theory is derived for a beam with a uniform cross section with the neutral axis near the centre of the cross-section. Gravity Dams, however, have a nearly triangular cross-sections, with its neutral axis at a different position - hence the method has proven to be conservative
- material limitations - foundation material properties, AAR effects, and variation in material properties cannot be modelled
- temperature loading cannot be simulated
- the method is limited to a 2D analysis - 3D effects cannot be taken into consideration and the dam cannot be analysed as a monolithic structure

- seismic loading cannot be accurately modelled

This method is well documented in many reliable sources such as USBR (1987), FERC (2002), USACE (1995), Kroon (1984) and Leclerc, Léger and Tinawi (2001).

The Classical Method is limited to doing static analyses. It can, however, be adjusted to include seismic analyses called "pseudo-static" (rigid body) and "pseudo-dynamic" analysis (explained briefly below).

For the purpose of so-called "pseudo-static" analyses, the earthquake load is treated as an inertial force applied statically to the structure. Westergaard's added mass principle is used to represent the hydrodynamic effects of the reservoir and tailwater. However, as Leclerc, Léger and Tinawi (2001:32) explains, the dynamic amplification of inertia forces along the height of the dam due to its flexibility is neglected. The inertial force split into two components; the inertial force of the concrete and the inertial force of the external hydrostatic loads (using the Westergaard formula). The following formulae are used to determine the inertia of the concrete and the additional water load due to the earthquake force according to the Westergaard's parabolic approximation formula:

$$P_e = W\alpha \tag{2.15}$$

$$p_{ew} = \frac{2}{3} C_e \alpha \gamma \sqrt{hy} \tag{2.16}$$

$$P_{ew} = \frac{2}{3} C_e \alpha \gamma h^2 \tag{2.17}$$

$$C_e = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1000t_e}\right)^2}} \tag{2.18}$$

Where:

P_e represents the resultant horizontal inertial earthquake force within the concrete
p_{ew} represents the hydrodynamic pressure applied to the face of the wall at the depth y under the water level
P_{ew} represents the resultant horizontal earthquake force acting at 0.4h
α =a/g is the seismic coefficient (usually assumed to be 0.1)
W is the weight of the dam
C_e dimensionless earthquake constant usually 0.82. This factor depends on the depth of water and the earthquake vibration period
h is the total water depth
γ is the specific weight of water
a is the dynamic acceleration and g is the gravitational acceleration
t_e is the period of vibration

The so-called pseudo-dynamic analysis is based on the simplified response spectra method described by Chopra (1988). This method is similar to the pseudo-static analysis method except that it recognises the dynamic amplification of the inertia forces along the height of the dam wall. However, the oscillatory nature of the amplified inertia forces is not considered (Leclerc, Léger & Tinawi, 2001:36).

It should be noted that the pseudo-static and pseudo-dynamic methods are considered to be the more approximate (and conservative) methods to simulate seismic loads. The structural response to random dynamic loading and the complex interaction between the natural mode frequencies and the frequencies of earthquake loads are not accounted for (Durieux, 2008:25).

Usually, if the Classical Method indicates that the dam is stable, no further analysis will be deemed necessary. However, if it fails, then more sophisticated methods should be used to confirm its instability. Safety factors for this method is presented in Section 2.7.

It should be noted that the Classical Method has received a great deal of criticism by academics and experienced engineers due to its many limitations. As explained by Durieux (2008:10), the use of the Classical Method does not always allow the engineer to reach an optimum solution for rehabilitation. In this case the FEM is a more accurate means of reaching an optimum solution.

2.6.3 The Finite Element Method

2.6.3.1 General

The FEM is a more complex method which can be used to design and analyse all types of concrete dams. Doing a full study of this method falls beyond the scope of this dissertation, therefore, the method will only be briefly discussed.

The FEM is based on the elastic continuum mechanics theory and follows a displacement (or stiffness) approach. The FEM is derived from the formulation of a simple spring and is given by the following formula:

$$\{F\} = [K]\{u\} \tag{2.20}$$

Where,

$\{F\}$ is the external force matrix
$\{u\}$ is the displacement matrix
$[K]$ is the global stiffness matrix

A geometric model of the system is defined by geometry (consisting of nodes and elements.), mechanical properties, boundary conditions, and loads which are applied to generate matrix equations for each element within the structure. These matrix equations are then assembled to create a global matrix equation for the structure. The equations are solved for the deflections. Strains, stresses and reactions are then computed using the results of the deflections. These results are stored and used to create graphic plots representing the results.

This method allows the engineer to closely model the actual (full) geometry of the structure and account for its interaction with its foundation. Thermal loads and AAR effects can also be included in the analysis. A notable advantage of this method is that it allows for the modelling of complicated foundations with varying material properties, weak joints on seams as well as fracturing. Varying material properties of the concrete itself can also be modelled.

Generally, for long conventional concrete dams with transverse contraction joints and without keyed joints, 2D analyses are appropriate and should be reasonably correct (USACE, 1995:5-1). The model should be assumed to be in plane strain. The engineer should however note that the most realistic structural response is three dimensional.

Furthermore, the USACE (1995:5-1) recommends that structures that are located in narrow valleys between steep abutments and dams with varying rock properties which vary across the valley should be analysed my means of a 3D model.

There are several commercial FEM software packages available. Some of the more popular packages used in industry include MSC Marc, MSC Patran and Nastran, Abaqus, ANSYS, ADINA, and DIANA. AutoCAD is generally used in combination with these programmes as a drawing tool to create the initial model.

2.6.3.2 Static and Dynamic Methods of analysis

The Finite Element Method can be used for both 2D and 3D linear and non-linear static as well as dynamic analyses.

Static analysis (FEM)

For linear and non-linear static FEM analyses purposes, the loads acting on the dam wall are computed in the same way as done for the classical method (as explained in Section 2.3). The non-linear static (Drucker Prager) method can be done as described by Durieux (2008:46-47) and is discussed in detail in his MEng dissertation. This method evaluates the safety of the structure in terms of material strength and simulates the yielding behaviour of the wall. Concrete material properties such as the tensile and compressive strength (f_t and f_c), modulus of elasticity (E), poison's ratio (v) and the material density (ρ)

are needed to conduct this type of analysis. As mentioned previously, this is a more reliable method of calculating the crack length.

Dynamic analysis (FEM)

As an initial estimate, Durieux (2015b:319) explained that the "pseudo-static" method (which is based on the classical method) can also be used to obtain as a fast solution. However, it is not recommended as a design or analysis standard. This approach is conservative. FERC (2002:3-22) reported that the pseudo-static analysis is not acceptable.

For a more realistic results, a modal analysis should be done. This method is performed to determine the major modes of vibration of the structure and the response of the structure to the earthquake is expressed as a combination of individual modal responses. Furthermore, FERC (2002:3-20) recommends two acceptable techniques to be used, namely the Response Spectrum and Time History (transient) methods. The dynamic analysis should start with a response spectrum analysis and progress to more refined methods if needs be (USACE, 1995:5-4). The time history (or transient dynamic) analysis is a more rigorous and precise method. The dynamic input data for this type of analysis is in the form of an accelerogram, which is a ground acceleration vs. time graph. The disadvantage of this method is the large amount of input and output data to be processed (Durieux, 2015b:319). This method is usually used for non-linear material dynamic analyses, when cracking is indicated by the response spectrum analysis. The response spectrum method is more popular and is recommended by many dam building organisations (Durieux, 2015b:319). The dynamic input data for this type of analysis is in the form of a ground response spectra, which is a ground acceleration vs. frequency graph.

For these methods the hydrodynamic loads can be simulated by Westergaard's "added mass" principle or the more complex fluid structure interaction (FSI). The Westergaard method is considered to be the most efficient and economical technique - and produces acceptable results. The FSI method, however, has proven to produce more accurate results as the compressibility of the fluid medium is taken into consideration.

As discussed by Vezi (2014:11-17), the Westergaard (1933) added mass theory can be used to model the hydrodynamic effects of the reservoir. The formula, according to this theory, for the hydrodynamic pressure (P_z) acting at height (z) from the base of the dam applied horizontally against the upstream vertical face is given by:

$$P_z = \frac{7}{8}\alpha\gamma\sqrt{Hz} \tag{2.21}$$

Where:

α is the horizontal ground acceleration

γ is the unit weight of water

H is the total height of the water

Furthermore, as discussed by Vezi (2014:57), Westergaard's theory can be applied in the FEM by applying nodal inertial mass loads at the wetted upstream face of the dam wall. These inertial mass loads can be calculated by multiplying the computed pressure at a given height (of the node) by the tributary area of the specific node - depending on its location.

As an alternative method, Durieux (2007:7) explains that the dynamic impact of the water on the dam wall can also be simulated by manipulating the density of the upstream elements in such a way that the so-called "added mass" (described by Westergaard (1933)) is included in the total weight of the concrete. According to Durieux (2007:7), this "density method" results in less stress disturbances on the upstream face. This method has frequently been used in the Department of Water and Sanitation to design and analyse dams and appurtenant structures (such as inlet towers).

$$b' = 0.38\sqrt{\mathrm{hy}} \tag{2.22}$$

$$\rho_i = \frac{w_i + b'}{w_i} \cdot \rho_c \tag{2.23}$$

Where:

b' represents the equivalent body of virtual concrete

ρ_i represents the density from the added mass theory incorporated in the first layer of upstream elements

ρ_c represents the density of concrete

h represents the depth of total water

y represents the depth below the water level

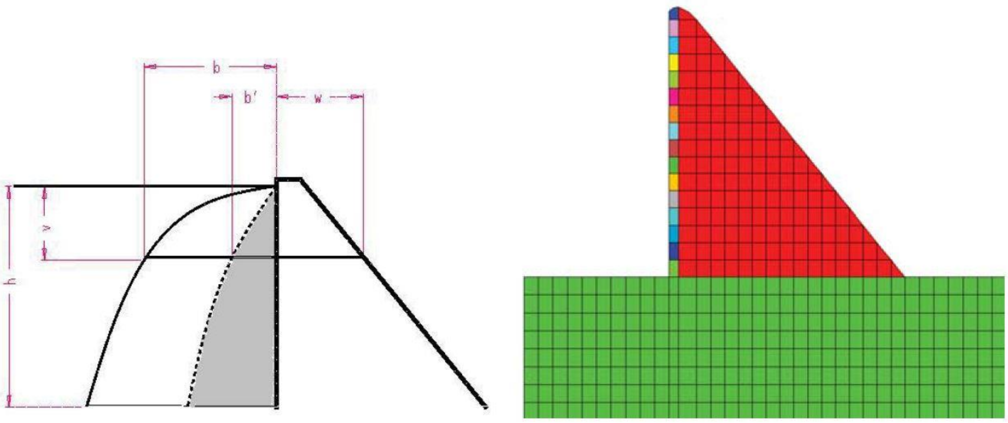

Figure 16: Diagram showing the Westergaard (1933) added mass theory and the varying densities on the upstream face of the finite element model (Durieux, 2007:8)

FERC (2002:3-20), explains that the purpose of performing a dynamic analysis is to determine the damage that will be caused during an earthquake so that this damage can be accounted for, and then determine if the dam can continue to resist the applied static loads in a damaged state.

2.6.3.3 Calibration of the model

It is important to have a FEM model which corresponds to the actual behaviour of the structure. For this purpose, both the static and dynamic models should be calibrated. In the static model can be calibrated by altering its material properties (i.e. manipulating its stiffness) until the FEM displacements approximately coincides with the actual measured displacements from dam monitoring readings (e.g geodetic surveys). Nevertheless, this should be done with caution as the calibration exercise might lead to unrealistic material properties. The dynamic FEM can be calibrated by adjusting the "added mass" and comparing the calculated natural frequency modes to the actual tested ambient vibration modes. A more representative and realistic model can be achieved by means of calibration, and in turn more accurate results can be produced.

2.6.4 Probabilistic safety evaluation of concrete gravity dams

2.6.4.1 General

According to Reynolds and Oosthuizen (2015:56), to find a dam "unsafe" is easy when using deterministic methods. The safety criteria is usually ultra-conservative and may result in the recommendation of unnecessary rehabilitation. Furthermore, they explain that probabilistic methods indicate "how safe" an existing dam is. They, therefore, emphasised that these probabilistic evaluations should be regarded as complementary to the traditional deterministic approaches.

The Leclerc, Léger and Tinawi (2001:46) explains that a probabilistic analysis considers explicitly the uncertainties in loading and strength parameters that are considered as random variables. These uncertainties are then transformed in the probability of failure of a dam. Probabilistic analyses require more information than deterministic analyses (e.g. the basic statistical properties of a data set of tested values are required).

Oosthuizen (1985) provides a guideline methodology for the probabilistic evaluation of dams in South Africa. This guideline is extensively used by the DWS in the safety evaluation of most existing dams in South Africa.

In this section only a brief summary of the probabilistic analysis of concrete dams in a South African context, as done in the DWS, will be discussed. The complex detail of probabilistic analysis falls outside the scope of this dissertation.

2.6.4.2 Probability of failure of concrete gravity dams

Oosthuizen et. al. (1991:119) explains that the probabilistic analysis can be performed on one of four levels, ranging from Level 0 to Level 3. The analysis procedures and assumptions increase from simple at Level 0 to advanced at Level 3 which is of more academic than practical importance. The problem to be addressed must be defined, the key aspects (in terms of the demands and resistance) of the structure should be identified, then a decision must be made as to which level of analysis will be performed.

Furthermore, they explain that to evaluate the probability of failure of a dam, the modes of failure, demands (or loads) on the structure, and resistance (its potential to resist the demands) of the structure must be identified. For gravity dams the modes of failure can be classified as sliding, overturning and other (foundation failures, earthquake failures etc.). Sliding and overturning failure are generally the more predominant modes. Other failure modes can be accounted for using relative probabilities established from past experience of dam failures with added site-specific adjustments by experienced engineers (Oosthuizen et al., 1991:128).

The cohesion (c) and angle of friction (ϕ) both affect the probability of sliding failure. For the probability of overturning, the tensile strength of the concrete (f_t), which can conservatively be assumed to be equal to the cohesion (c), is the core parameter. Floods (i.e. water level) affects both probabilities.

The total combined probability of failure can be determined by using the following formulae:

Total probability of sliding or overturning:

$$p(Sliding\ or\ overturning\ failure) = p(sliding) + \ p(overturning) - p(sliding)(overturning) \tag{2.24}$$

As mentioned previously the probability of “other failure modes” can be incorporated by using relative probability. Oosthuizen et. al. (1991) and Oosthuizen (1985) both discuss the estimation of the probability of failure of gravity dams in great detail and provide some practical examples.

2.7 LOAD COMBINATIONS AND FACTORS OF SAFETY

Durieux (2008:19) explains that there are no standardized codes for the design and analysis of gravity dams. Only guidelines exist, therefore, it is up to the APP and the review panel to decide on the criteria to be used. For this reason, the guidelines for the load combinations and safety criteria presented in this section may be modified at the engineering discretion of the APP and his team.

2.7.1 Load combinations and stability criteria for the Classical Method

Table 1 presents typical load combinations for the analysis of gravity dams used in South Africa, as recommended by the Department of Water and Sanitation (DWS) - previously known as the Department of Water Affairs and Forestry (DWAF).

Table 1: Typical load combinations used for the Classical Method to design gravity dams (Oosthuizen, 2006)

Load Combination	Hydrostatic							Other loads			
								Uplift		Earthquake	
	MOL	FSL	RDF	SEF	TW	S	G	Partial	Full	OBE	MCE
Service-1			X		X	X	X	X			
Service-2	X					X	X	X			
Abnormal - 1		X			X	X	X	X		X	
Abnormal - 2			X		X	X	X		X		
Extreme - 1		X			X	X	X	X			X
Extreme - 2				X	X	X	X	X			

Where:

MOL	Minimum Operating Level
FSL	Full Supply Level
RDF	Recommended Design Flood
SEF	Safety Evaluation Flood
TW	Tailwater
Partial Uplift	When a drainage gallery is present and drainage criteria are specified
Full Uplift	Uplift level is equal to FSL in the on the upstream and tailwater level on the downstream side
G	Gravity
OBE	Operating Basis Earthquake
MCE	Maximum Credible Earthquake

The service (or usual) load combination incorporates the loads acting on the structure under normal operating conditions. Abnormal (or unusual) loads have a higher risk than service loads. The extreme load condition considers extreme/disaster occurrences. For this load condition, some degree of damage is brought into consideration, with a limitation that no catastrophic failure should occur.

Loads which are not included (such as ice, wind, temperature loads and so forth) should be considered in cases where they are applicable - at the discretion of the APP and his team.

Table 2: Typical stress ranges and safety factors used for gravity concrete section (Oosthuizen, 2006)

Load Combination	Tensile stress (MPa)	Compressive (MPa)	$FOS_{Sliding}$		$FOS_{Overturning}$
			Peak	Residual	
Service-1 Service-1	0.0	3.0	3.0	1.5	1.5
Abnormal-1 Abnormal-2	0.0 - 0.5	3.0	1.5	1.1	1.2
Extreme-1	0.2 - 1.0	3.0	1.3	1.0	1.1
Extreme-2	0.3 - 1.0				

Table 2 presents typical stress ranges which serves as a guideline for the design of concrete gravity dams using the Classical Method as recommended by the DWS.

2.7.2 Load combinations and stability criteria for the FEM

The current practise used in the classical method can be used as a basis for establishing the load combinations used for the FEM analysis. As mentioned previously, overturning and sliding failure can be triggered by the failure of the materials of the dam or foundation. Hence, for the FEM analysis, the computed stresses within the dam and/or its foundation should not exceed their respective material strengths.

Furthermore, as FERC (2002:3-19) explains, it is ultimately up to the reviewers to determine the value of the analysis based on how it addresses the possibility of the identified failure mechanisms.

2.8 LEGISLATIVE REQUIREMENTS FOR LARGE DAMS

The Dam Safety Regulations (published in Government Notice R. 139 of 24 February 2012) has additional requirements for dam safety evaluations of category III dams. All large dams (dams with a height of 30 m and higher) are classified as category III dams. See Appendix A for classification of dams with a safety risk.

According to Section 36(1) of Government Notice R. 139 of 24 February 2012:

(1) *The requirements and conditions set out in regulation 35 in respect of a dam safety evaluation of a Category II dam, also apply to an evaluation of a Category III dam, except that –*

(a) *the dam safety evaluation and on-site inspection must be carried out by an approved professional person assisted by a professional team;*

(b) *the information required by sub-regulation 35(6)(h) must also include characteristic results obtained in the process of evaluation in terms of subregulation 35(4)(d);*

(c) *a dam safety risk analysis and/or risk assessment must be carried out on the dam and an indication of the probabilities provided, when requested by the Director-General; and*

(d) *the members of the professional team must sign the relevant sections of the report for which they are responsible.*

The National Water Act (1998) defines an approved professional person (APP) as:

> *"a person registered in terms of the Engineering Profession of South Africa Act. 1990 (Act No. 114 of 1990), and approved by the Minister after consultation with the Engineering Council of South Africa"*

According to Government Notice R. 139 of 24 February 2012 the professional team is:

> *"one or more persons with expertise in disciplines in which expertise is required, and which disciplines have been determined by the approved professional person concerned with the concurrence of the Director-General, and who after submission of particulars of their names, qualifications and professional experience have been approved by the Director-General;"*

Furthermore, Section 16(2) it states that:

> *" In the case of a dispute or if the Director-General is of the opinion that -*
>
> > *(a) a project is an extraordinarily large one;*
> >
> > *(b) unusual design principles or methods of analysis have been used;*
> >
> > *(c) unusual construction procedures or construction materials have been specified;or*
> >
> > *(d) extraordinary circumstances exist;*
>
> *he or she may require that the owner in respect of the proposed project appoint an independent expert or team of experts to evaluate the proposed design, drawings, specifications, anticipated circumstances during construction of the dam or first filling of the reservoir, in whole or in part or any other aspect thereof, and submit a report on its findings to the Director-General."*

In summary, for the evaluation of the overall safety of a Category III dam, the APP is required to be assisted by a professional team. Professional team members must be approved to assist the APP in specific fields, for example, structural analysis, geology, monitoring system, flood hydrology, outlet works, etc. In some cases, a professional team member may not be necessary in a specific field if the APP is considered to have sufficient expertise in that field. Both the APP and the assisting professional team must be approved by the relevant authority as mentioned above. In the case of a dispute, an independent expert or team of experts should be appointed to evaluate the findings. Their appointment should also be approved by the relevant authority.

2.9 CHAPTER CLOSURE

This chapter presented a concise background of gravity dams, their potential failure modes and some historical failure incidents, an insight to common structural safety evaluation methods, as well as some legislative requirements for the evaluation of the safety of large dams in South Africa. The next chapter presents a case study i.e. the structural safety evaluation of Nqweba Dam.

CHAPTER 3 – CASE STUDY: STRUCTURAL SAFETY EVALUATION OF NQWEBA DAM

3.1 BACKGROUND

3.1.1 Location of the structure

Nqweba Dam, previously known as Van Reyneveld's Pass Dam, is located on the Sundays River, approximately 1 km north of the Graaf-Reinet in the Eastern Cape. The dam is situated in the Camdeboo National Park and is thus a tourist attraction in the Graaf-Reinet area. Figure 17 shows the geographic location of Nqweba Dam.

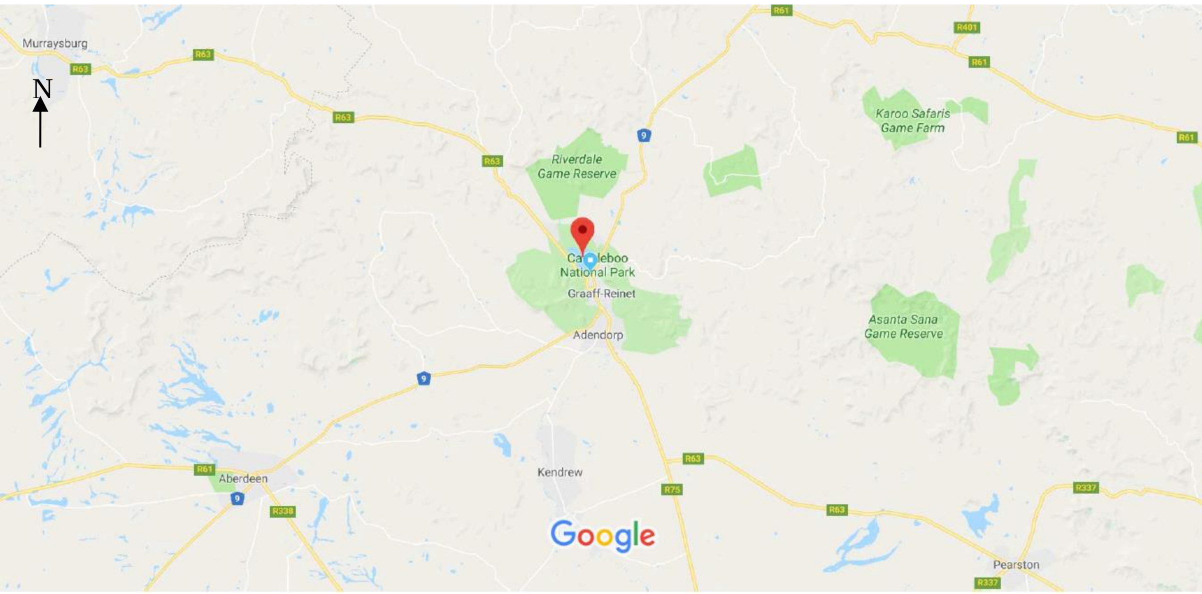

Figure 17: Geographic location of Nqweba Dam (Google Maps, 2018)

3.1.2 Brief history of the dam

General

Nqweba dam was designed and built by the Irrigation Department which is known today as the Department of Water and Sanitation. Construction of the dam began in 1921 and was concluded in 1925. The dam wall is a concrete gravity structure, with a unique stepped downstream face finish. Downstream views of the dam are presented in Figure 5 (in Section 2.2.2) and Figure 18 (during overspilling). The steps dissipate the energy of the water over spilling the structure to reduce the impact of the water and erosion on the toe and foundation immediately downstream of the dam.

Figure 18 : Downstream view of Nqweba dam during overspilling, presumably in 2008 (Dam 2008, n.d)

The wall has a maximum height of approximately 46 m at its deepest section. And it has a crest length and width of 381 m and 3 m, respectively. Over the years, the dam has lost a significant portion (approximately 43%) of its capacity as a result of siltation. It currently mainly provides water to the Camdeboo Local Municipality, for domestic purposes. The dam is classified as being a large category III dam with a high hazard potential (see Appendix A for more information about dam classifications). The table below summarises some general information of Nqweba Dam.

Table 3 : Summary of general information of Nqweba Dam.

Type	Concrete gravity with an uncontrolled ogee spillway
Completion of construction	1925
Classification	
• Size	Large
• Hazard potential	High
• Category	III
Height above riverbed (height in terms of regulations)	33.45 m
Height at deepest section	46.33 m
Mean Annual Precipitation (MAP)	350 mm
Purpose	Domestic water supply
River	Sundays
Original storage capacity (1925)	79.03×10^6 m^3
Current storage capacity (2011)	44.72×10^6 m^3 (43.42 % silted)
Non-overspill crest (NOC) level	RL 790.35 m
FSL	RL 787.58 m
Total spillway length	199.16 m
Spillway capacity	1 650 m^3/s
Catchment size	3668 km^2

Recommended Design Flood (RDF)	4 770 m^3/s
Safety Evaluation Flood (SEF)	8 200 m^3/s

Geology

According to the resident engineer during construction (Mr. K.R Shand), most of the foundation rock exposed by excavation was quite sound. The rock samples of boreholes drilled through the dam and into its foundation (in 1987) also confirmed that Nqweba dam is generally founded on hard dolerite bedrock of excellent quality. Shand (1924:17) reported that the dolerite practically has a slight upstream dip throughout the foundation. Furthermore, he reported that the rock is traversed by a few vertical seems containing paler dolerite and in some cases decomposed dolerite. The rock samples generally demonstrated unweathered, medium to widely jointed, tight rock and the contact between the concrete and foundation was found intact – except for far up on the left flank, next to the unlined spillway (Shall, 1988:2-3). See *Figure 1: Positions and geological columnar sections of boreholes on dam wall* from Shall (1988) in Appendix B.

Shand (1924) reported that "*the only serious trouble was encountered in the river section, where a large pocket of decomposed dolerite was struck; this was taken out until sound rock was found, which was at a depth of 42 ft.* (12.8 m) *below the river bed level*". Hence, the dam wall is buried quite deep into its foundation (up to approximately 13 m in the central "river" section). Schall (1988:4) deemed the foundation rock in the river and right flank to be sound.

During foundation excavation on the left flank, a significant amount of decomposed dolerite was encountered. This rock was seamy and broken up by fine vertical joints containing clay and decomposed dolerite (Shand, 1924:17). Shand (1924:17) reported that this rock was all removed, and the excavation carried well into the hillside. It was also mentioned that the excavation was not taken out to the full section, but a narrow cut-off trench was created. Schall (1988:4), expressed some concern with regard to the quality of the rock on the left flank and recommended that more detailed geotechnical investigations be done in this area. However, this is quite high up on the left flank (between the actual dam and the auxiliary spillway), and it is deemed unlikely that this area could have a major significance on the stability of the structure. Also, during construction excavation to sound rock in this area was obviously not considered to be necessary – and the material was also not removed to the full width of the excavation.

The dam is situated in an area of low seismicity (see Figure 19). Seddon et al. (2008:24) reported that the possibility of earthquake damage is too remote to be of any concern. They also reported that the dam has performed well since construction and no indication of any foundation movement or attributable distress of any kind was observed. It should also be noted that the dam safely withstood the 1974 flood of 3 470 m^3/s (which was the highest flood recorded to date).

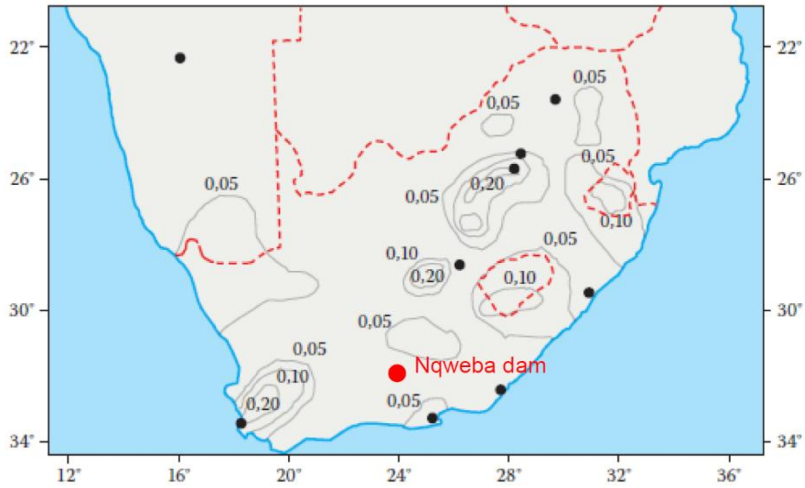

Figure 19 : Approximate location of Nqweba dam on a seismic hazard map showing peak ground acceleration in g with a 10% probability of exceedance in 50 years (South African Bureau of Standards [SABS] 0160:1989, Rev. 1993)

Concrete quality

Geertsema (1987) examined the concrete samples of boreholes drilled through the dam and into its foundation (in 1987). From these samples two types of concrete were identified. The first (Class I) is the main type of concrete, of which the mass of the dam was constructed. The second (Class II) was used for the construction of the bridge. Class I is of medium to high quality and Class II is of very low quality (Geertsema, 1987). Furthermore, Geertsema (1987) reported that no alkali reaction was found to have taken place in the concrete.

3.1.3 The safety concern

It is known that a number of concrete gravity dams (built before the early 1900's) were constructed without the consideration of the effect of uplift pressures. It is apparent that this is the case at Nqweba dam as no galleries, grout or drainage curtains are present in the existing structure.

According to the first and second Dam Safety Evaluation (DSE) reports submitted in 1998 and 2008, Nqweba dam does not comply with modern stability criteria for concrete gravity dams (Seddon et al., 1998 and 2008). The findings of these dam safety inspection reports led to the dam being put on top of the DSO's priority list of dams that have shortcomings and are in need of rehabilitation work to improve their safety.

Typically, this would mean that the dam must be rehabilitated to improve its safety and millions spent to do so. Nevertheless, some engineers believe that this is not the case and that major rehabilitation is not necessary.

3.2 AVAILABLE INFORMATION AND STUDIES

3.2.1 General

The latest DSE (Seddon et al., 2008) report stated that "*Extensive stability analyses have been carried out. The conclusion of these is that the dam fails to meet accepted stability criteria. There is a possibility of the dam failing in the event of a Recommended Design Flood (RDF) and a probability that it would fail in the event of a Safety Evaluation Flood (SEF), if these large floods were ever to occur. It is considered very unlikely that the dam would fail on a sunny day with the water at Full Supply Level (FSL).*"

Furthermore, Seddon et al. (2008) stated that "…*it is recommended that structural measures are undertaken to make the dam safe, or that the dam be decommissioned and replaced with another source of water.*"

It is very important to note that these reports both used the classical method of analysis (which is usually conservative) to evaluate the structural safety of the dam.

Engineers from the Dam Safety Surveillance sub-directorate of the DWS (amongst others) have done several investigations contradicting both Seddon et al. (1998 & 2008) and the DSO's views.

3.2.2 Probabilistic Analyses

Three risk analyses have been previously carried out for the dam, namely a conservative initial Level 1 risk analysis by Van der Spuy (1987), a Level 2 risk analysis by Van der Spuy (1989) and a Level 2 risk analysis by Van der Spuy (1992). These analyses were done using the guidelines provided by Oosthuizen (1985). They were also all approved by Dr. Oosthuizen who was the Chief Engineer of the Dam Safety Surveillance sub-directorate at the time (and also an APP for many large dams).

1987 Risk Analysis

This basic, Level 1 risk analysis was done with very conservative assumptions with regard to the material properties of the concrete, foundation and loads acting on the dam. At the time of this analysis the actual material properties were still being determined. The worst possible loading and the lowest assumed material properties were used for this analysis. Under these conservative assumptions, Van der Spuy (1987) found that the level of risk of the dam was classified as being unacceptable. The total probability of failure for the dam was calculated to be 0.75. It was recommended that urgent attention be given towards an emergency plan for the warning and evacuation of the community downstream of the dam in the case of the occurrence of a flood. Furthermore, Van der Spuy (1987) recommended that the determination of the actual material properties be given the highest priority so that a Level 2 risk analysis can be done for the dam – using the actual material properties.

1989 Risk Analysis

In 1989, Van der Spuy performed a Level 2 risk analysis for the dam. In this (Level 2) analysis, the total probability of failure of the dam was calculated to be lower than $1x10^{-4}$ (compared to 0.75 in the Level 1 analysis). Van der Spuy (1989:21) explained that the purpose of a Level 1 analysis is to use available information of the dam in together with conservative assumptions about unknown information to determine if a more in-depth analysis is necessary. Its purpose is not to attempt to obtain an absolute value for the probability of failure of the dam. Hence, a more detailed risk analysis was deemed necessary. Furthermore, the Level 1 risk analysis should determine what additional information is needed to obtain more accurate results (i.e. actual material properties) and also what can be done to lower the level of risk of the dam while the Level 2 risk analysis is being done.

The purpose of the Level 2 risk analysis performed by Van der Spuy (1989:21) was to determine the probability of failure and level of risk of the dam as accurately as possible, so that decisions can be made with regard to the safety and need for rehabilitation work at the dam. For this risk analysis, the material properties as determined by laboratory tests (Maschek & Geertsema, 1988) and, where necessary, adjusted by values determined statistically. Hence, more realistic assumptions were used.

Van der Spuy (1989:22) concluded that:

- The level of risk for the dam is medium to high, which is considered to be acceptable.
- The weathered foundation on the left flank is not considered to have a detrimental influence on the safety of the dam wall. This weathered rock is localised in the region of the auxiliary spillway and a failure of the auxiliary spillway during a flood is preferred over a failure of the dam wall.
- The erosion of the downstream toe of the dam is due to normal weathering over time and does not endanger the dam.

Furthermore, Van der Spuy (1989:22) recommended that:

- Since the probability of failure for the dam falls in the lowest failure region for large dams, betterment works of the wall will have little influence on the level of risk of the dam. No rehabilitation work is therefore recommended.

- The level of risk of the dam in terms of human lives is abnormally high and an effort should be made to reduce it. The easiest way to do so is to set up a comprehensive emergency plan for the warning and evacuation of the people downstream of the dam.

1992 Risk Analysis

Lastly in 1992, Van der Spuy (1992) performed an evaluation of alternatives for improvement of Nqweba Dam by means of a risk-based analysis. These alternatives for improvement are basically ways of lowering the level of risk at the dam. The following alternatives for improvement were evaluated:

1) *Leaving the dam as is (doing nothing)*
2) *Widening the bridge deck and balustrades over the wall*
3) *Same as alternative 2: however, including the replacement of the broad-crested structure in the auxiliary spillway with a crump- structure*
4) *Lowering the auxiliary spillway but maintaining the capacity of the dam with "hydroplus"-sluice gates or "Mirza" radial gates*
5) *Provision for post tension anchor cables in the dam wall*
6) *Warning system for early evacuation of the area downstream if required*

After the evaluation of these alternatives it was concluded that the only option that is economically justified is alternative 6. Thus, Van der Spuy (1992) recommended that an effective warning system and emergency preparedness plan should be established to limit loss of life during the occurrence of a flood. He also motivated that it is also the only alternative that has a significant influence on the level of risk with regard to big natural floods. Besides ongoing monitoring of the condition and behaviour of the structure, the report recommends that the dam be left as it is.

Due to the extent of the probabilistic analyses already done for the dam, it was deemed unnecessary to execute another probabilistic analysis. The results of the analysis were reviewed and are still deemed valid for this case study.

3.2.3 Academic study by Cai (2007)

In his PhD dissertation, Cai (2007) developed a crack modelling (Non-linear Fracture Mechanics [NLFM]) technique which was successfully used in evaluating the safety of an existing concrete gravity dam in South Africa (Nqweba dam). The cracking response of the dam structure under static loading was also adequately predicted (Cai, 2007:2).

This analysis revealed that the dam is safe under FSL, RDF and SEF conditions. Furthermore, the results of his analysis revealed that the dam will be able to pass a maximum overflow of 17 m before it fails.

And the dam can be regarded as safe against sliding (shear) with an overflow water level of up to 20 m (Cai, 2007:200).

Cai (2007:202) does, however, mention that "*It is also worth pointing out that the water pressure that develops as cracks grow has not been taken into account in this research (or developed NLFM method). Therefore, the findings with regard to the safety of the dam should be taken as the maximum possible (upper boundary) safety limit that the dam can have.*"

3.2.4 Academic study by Durieux (2008)

Mr Hans Durieux, a DWS Specialist Engineer, examined Nqweba Dam as a case study as part of his master's dissertation in 2008. He performed a non-linear static finite element analysis for the dam (in 2D). From the results of the FEM analysis he concluded that: "*the dam would not fail due to material yielding even if the material strength* (tensile) *were to be as low as 1.5 MPa. The calculations also reveal that the risk of the dam suffering a vital crack that could cause failure is relatively small. For this dam the critical risk is sliding.*" However, high factors of safety against sliding (of approximately 8) were calculated using the actual tested material properties. In Durieux's opinion the tested material properties are too high.

Nevertheless, Durieux (2008:148) also mentions that the focus of his dissertation is on Drucker-Prager material strength and yielding behaviour and that it is not a comprehensive dam safety evaluation.

3.2.5 Instrumentation and monitoring

3.2.5.1 General

Naude (2014) reported that, in 1988, ten pore pressure meters and two sliding micrometer systems were installed at Nqweba dam. Due to the fact that the dam is heavily silted, these instruments were installed in order to obtain an understanding of the effect that the high silt loading has on the dam wall structure. The results of these monitoring systems are discussed briefly in this section.

3.2.5.2 Pore pressure meters

A pore pressure meter (also known as a piezometer), is a hydrostatic pressure measuring instrument which can be used to measure the pore pressures at various points within a dam wall and its foundation. This information would be very useful in terms of knowing what the actual uplift pressures are within a dam and its foundation.

Unfortunately, the ten pore pressure meters that were installed at the dam were vandalised. The connecting cables and readout units were stolen before any meaningful results could be officially recorded. This is vandalism occurred during a period when there was no security at the dam site to prevent unauthorised access to the dam and installed instrumentation (Naude, 2014). Fortunately, the Sliding Micrometer systems were not harmed and are still in full operation.

However, Seddon et al. (2008:24), reported that Dr Oosthuizen stated that piezometers installed in the concrete showed zero pore pressures. This is worth bearing in mind but should be verified by obtaining actual recorded readings.

3.2.5.3 Sliding Micrometer systems

The Sliding Micrometer is a measuring system which takes high-precision, to the measuring line, axial displacement measurements for boreholes and measuring lines in rock, concrete or soil in any arbitrary direction (Solexperts, 2015:6).

A Sliding Micrometer system fundamentally consists of:

1. an unplasticized polyvinyl chloride (uPVC) measuring tube with measuring points at 1m intervals installed in vertical borehole drilled from the non-overspill crest (NOC) of the dam, through the dam body and into its foundation,
2. a Sliding Micrometer probe (with a winch, guiding rods and chain), and
3. a readout unit (a laptop with battery set-up).

The process of taking readings essentially consists of lowering the probe to the bottom of the hole, then hoisting it and recording readings at each consecutive measuring point in the measuring tube.

According to Solexperts (2015:2), the accuracy of measurement of the Sliding Micrometer is better than ± 0.002mm/m. The position of the two Sliding Micrometer systems (namely, Vrvm 1 and Vrvm 2) installed at Nqweba Dam is presented in Figure 20 (downstream view). Vrvm 1 has 39 measuring points, and Vrvm 2 has 51.

The initial “zero” readings were taken in summer 1988. Thereafter, readings were taken on a bi-annual basis (once in winter and once in summer). To detect movement between measuring points over time, the “zero” readings are deducted from all future readings and the results plotted on a graph. Both the relative and cumulative (z) displacements are plotted on graphs. All displacements are relative to the bottom measuring point (Naude, 2014).

For the period of 1988 to 2014, Naude (2014) reported that the vertical displacements measured were within acceptable and safe limits. Furthermore, for this period Naude (2014) concluded that, based on displacements measured by the two systems, no signs of uplifting and/or expansion more than expected due to seasonal temperature fluctuations were detected. See Appendix C for the plotted results for this period from the report compiled by Naude (2014).

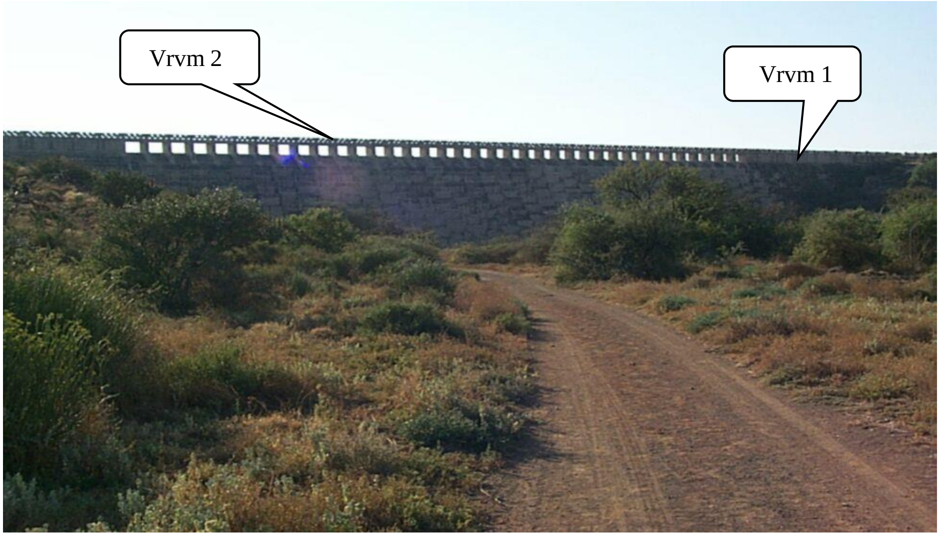

Figure 20: Approximate location of the two Sliding Micrometer systems installed at Nqweba Dam (Naude, 2014)

For some time (2014 to 2016) the author was responsible for taking these readings on-site as well as processing and analysing these readings. The Sliding Micrometer monitoring results up to the end of 2016 are presented in Figure 21 and 22. An evaluation by the author of the Sliding Micrometer monitoring readings up until the end of 2016 is summarised below.

Vrvm 1

The relative summer readings are mostly within 100 microns of contraction (shortening) and expansion. An exception is noted at 4-5 m below the NOC, where a possible joint is located. Contraction (cumulatively of up to approximately 2 mm) is evident in the top 14-15 m of the wall during winter periods. This could be linked to the fact that vertical joints are only present on the upper portion of the dam wall. The relative winter readings are within 100 microns of expansion and 250 microns of contraction. Most of these trends have been noted previously and should be closely monitored to determine whether or not they continue. Nevertheless, the vertical (Z) displacements for this system are considered to be within normal and acceptable limits for this dam.

Vrvm 2

For high water levels, contraction (of approximately up to 1.5 mm) is evident in the top 14-15 m of the wall during winter periods. Just like in the Vrvm 1 system, this could be linked to the fact that vertical joints are only present on the upper portion of the dam wall. However, some expansion during winter periods is noticed in the lower part of the dam wall (approximately between 15 m and 30 m). The difference between high and low water levels is more evident in this system for both winter and summer readings. In this area the dam wall seems to expand when water levels are low and contract when water levels are higher. Signs of a possible joint is evident at 7m below the NOC.

Most of these trends have also been noted previously and should be closely monitored to determine whether or not the they continue. The vertical (Z) displacements for this system are also considered to be within normal and acceptable limits for this dam.

The author confirms the findings and conclusions made by Naude (2014). An important addition to his findings is that the seasonal expansion and contraction seen mostly in the upper part of the wall can be linked to the presence of vertical joints in this region.

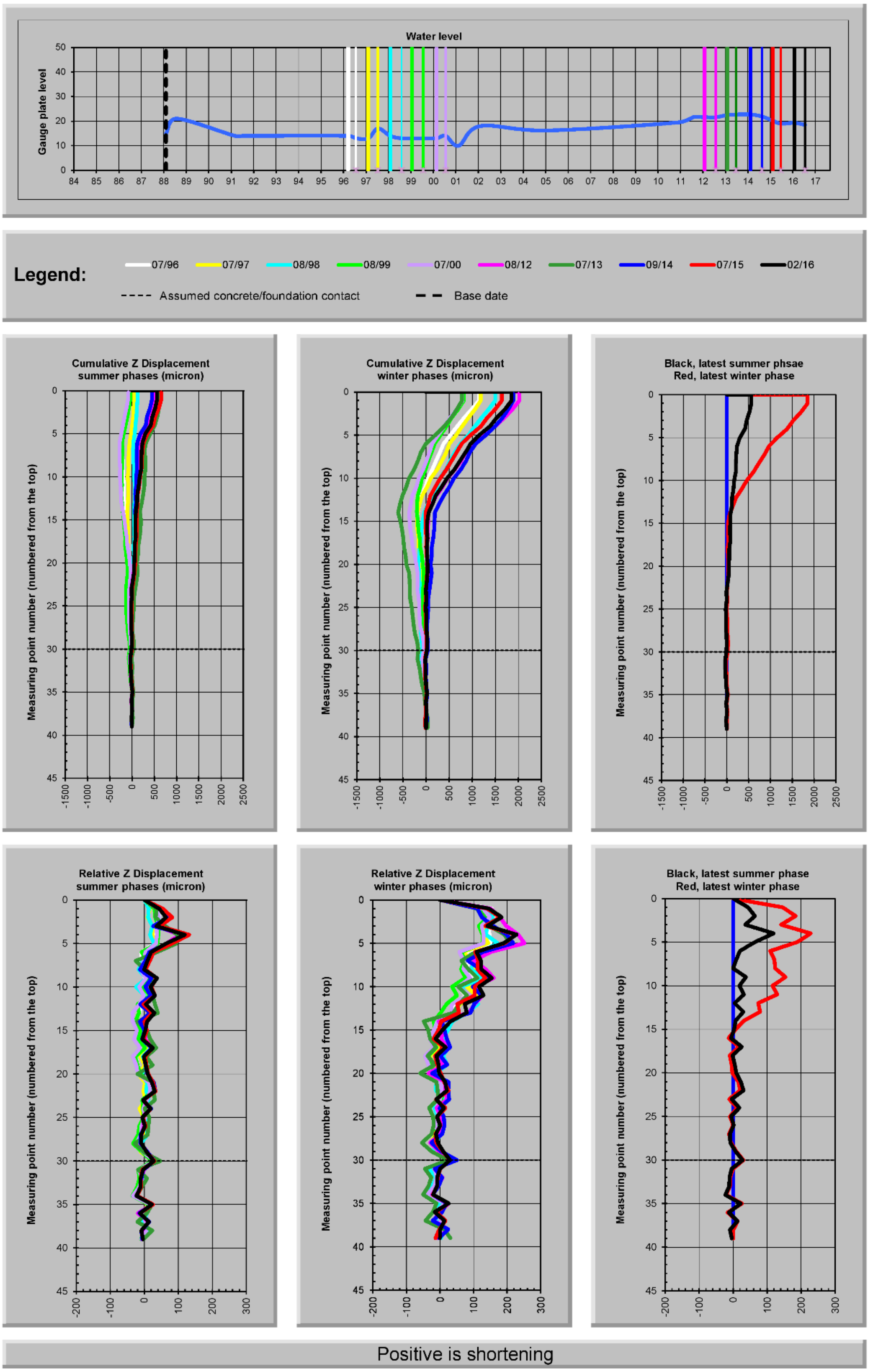

Figure 21: Sliding Micrometer readings for Nqweba dam up until Winter of 2016 (Vrvm1)

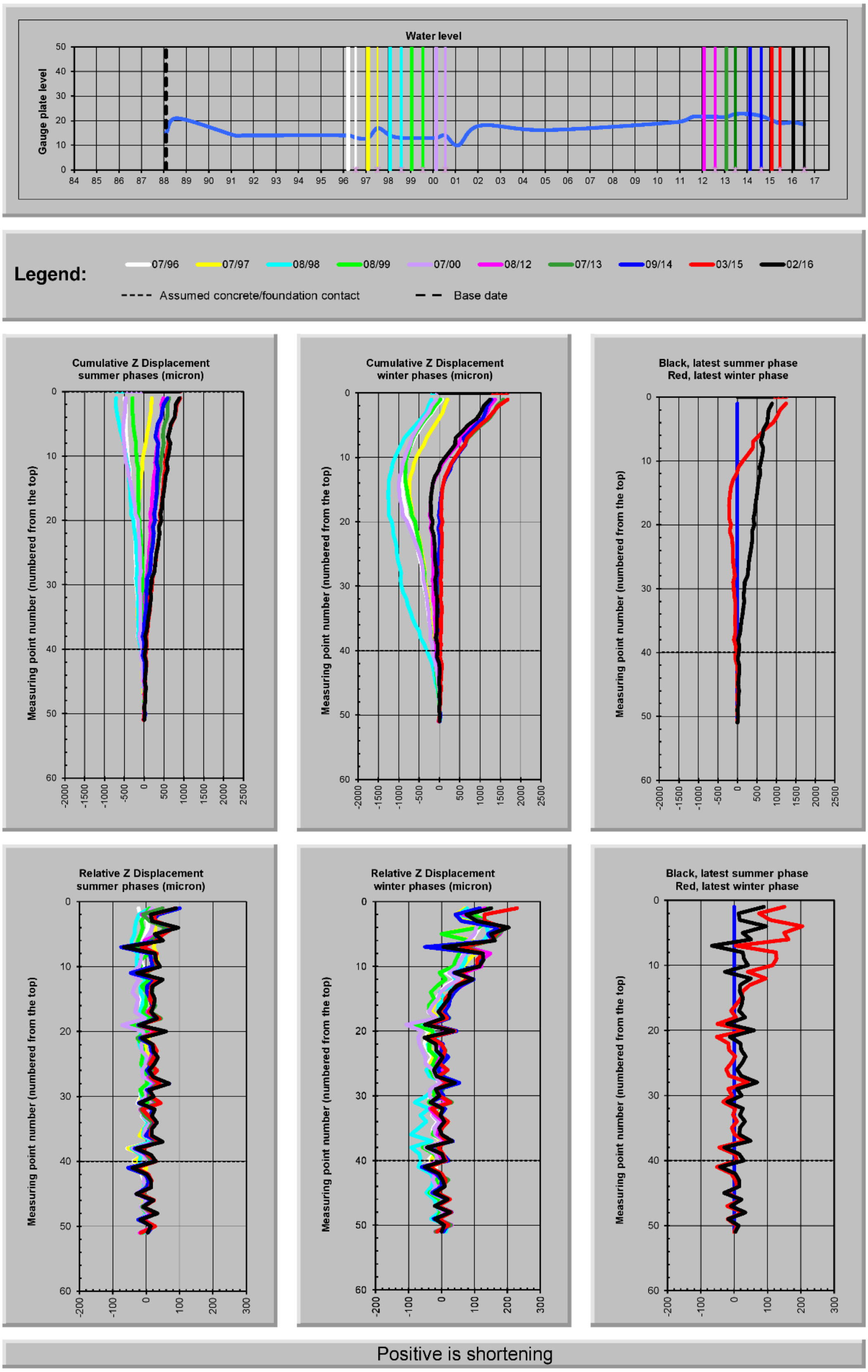

Figure 22: Sliding Micrometer readings for Nqweba dam up until Winter of 2016 (Vrvm2)

3.2.6 Other possible significantly stabilising factors

3.2.6.1 Reservoir silt causing a reduction in uplift pressure

According to FERC (2002:3-9), reservoir silt may reduce uplift pressures under a dam in a similar manner to that of an upstream apron. Figure 23 illustrates how uplift forces may be reduced by silt. It is known that Nqweba dam is heavily silted (approximately 43%). Seddon et al. (2008:21) also reported that the silt at Nqweba dam has been shown to be very stiff - which reinforces this very idea. The silt layer is also very thick, covering most of the dam wall height, with only approximately 9 m of water above it to the FSL.

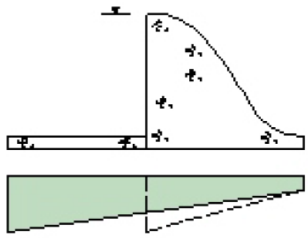

Figure 23: Reduction of uplift under dam due to upstream apron - or similarly reservoir silt (FERC, 2002:3-9).

The thick layer of silt at Nqweba dam may well prevent full uplift forces from developing. The extent of uplift reduction should be verified by instrumentation and monitoring.

3.2.6.2 Frictional resistance on dam wall caused by silt

It has been suggested by some dam engineers that, downward drag (frictional resistance) caused by the high level of silt on the upstream face could have a stabilising effect on the stability of the structure. This resistance has been taken into account in the risk analysis done by Van der Spuy (1989:16). It is noted, however, that it is not common practise to incorporate this force in conventional stability analyses for concrete gravity dams.

3.2.6.3 Stabilizing arching effect and "keying of jointing" in bottom portion of the dam wall

On the progress of construction drawing (See Appendix B, drawing 130403/98), it can be seen that vertical contraction joints were only included above RL 768 m. Below this level, there were long pours of up to approximately 87 m with stepped and keyed vertical construction joints. It can be seen that horizontal joints are also stepped and keyed.

Seddon et al. (2008:22), reported that "staggering and keying" of the jointing in the lower part of the structure should have a beneficial effect with regard to any horizontal stabilizing action that may occur. Furthermore, they mention that it was suggested by a senior specialist engineer from the DWS that horizontal arching action across the valley should be investigated as a possible means of preventing failure of the dam.

The monolithic and three-dimensional (3D) behaviour of the structure are not taken into consideration in the conventional (2D) analyses. These effects can only be incorporated by performing, more advanced, 3D FEM analyses.

3.2.6.4 Conclusion with regard to the additional stabilizing factors

It is not common practise to incorporate these possibly stabilizing factors in stability calculations. It is also difficult to accurately estimate their extent, as it may vary from site to site. Ultimately, they are unknowns which can only be legitimately verified by comprehensive instrumentation and monitoring.

3.3 THE FEM ANALYSES

3.3.1 General

The latest two DSE reports used the simpler classical/conventional method to analyse the safety of the structure. Despite its popularity, this method of analysis has many limitations and produces conservative results – as discussed in the Chapter 2.

Therefore, based on the review of available information (and studies) and for the purpose of this investigation it was deemed necessary to perform a more advanced type of analysis, namely, a FEM analysis. It is believed that the FEM analyses will provide a more realistic insight into the behaviour of the structure. It will also assist in confirming the status of the dam's structural integrity. The decision to perform FEM analyses is also very well in accordance with USBR (1987:330) recommendations (see Section 1.1).

The following FEM analyses were performed for both 2D and 3D models of the dam:

- Static - Linear Elastic
  - Non-linear Elastic ("Drucker Prager")
- Dynamic - Modal Analysis
  - Response Spectrum Analysis for OBE and MCE earthquake loading

The 2D FEM analyses will be performed to achieve a more realistic evaluation of the safety of the structure (compared to the well-known and commonly used classical method). The 3D FEM analyses will be done for the entire dam structure to analyse the dam as a monolithic structure and to take possible horizontal arching action of the lower part of the structure across the valley into account. As a result of taking these factors into account and performing advanced structural analyses, it is believed that a more realistic evaluation of the safety of the dam will be achieved.

It is known that the dam is located in an area of low seismicity – so earthquake damage is unlikely. The main purpose of the performing dynamic FEM analyses is to determine the mode shapes and natural frequencies of the structure. This could be useful for long term structural health monitoring of the dam.

Software used

The following three software programs were used to perform the FEM analyses:

- AutoCAD Civil 3D,
- MSC Patran, and
- MSC Marc

AutoCAD Civil 3D was used to create the geometry in the form of a wire mesh. The wire mesh was then exported to MSC Patran, which was used to transform the wire mesh into a solid, create an initial finite element mesh. The model was then exported to MSC Marc, which was used to apply material properties, boundary conditions, create load cases, apply loads and perform the computations of the analyses.

3.3.2 Material properties

The concrete of the dam wall is still in a very good condition. Laboratory tests indicated that the tensile stress is still reasonably good, even some 62 years after construction (Van der Spuy, 1992:D4). Table 4 presents the tested material properties:

Table 4: Tested material properties (Van der Spuy, 1992:D2-D4)

Concrete	
Density (ρ_c)	2455 kg/m^3
Compressive strength (f_c)	30.2 MPa
Tensile strength (f_t)	3.07 MPa
Modulus of elasticity (E_c)	28 000 MPa

The following material properties were accepted and used in the FEM analyses:

Table 5: Accepted and used material properties

Concrete	
Density (ρ_c)	2455 kg/m^3
Poisson (ν_c)	0.2
Modulus of elasticity (E_c)	28 000 MPa
Foundation	
Density (ρ_r)	0 (standard practice)
Poisson (ν_r)	0.22
Modulus of elasticity (E_r)	30 000 MPa

Silt		
Unit weight (γ_s)	3.6 kN/m^3	
Soil properties for Rankine's theory		
Parameters	Upstream silt	Downstream earth
c	19.3 kPa	0 kPa
ϕ	23.8°	33°

To evaluate the integrity of the structure in terms of material strength and simulate the yielding behaviour of the wall, the non-linear Drucker Prager yield model was used.

The tested tensile and compressive strength can be used to estimate the c and φ values needed to calculate the Drucker Prager parameters used in the non-linear analyses. The table below presents the calculated Drucker Prager parameters using the actual tested concrete properties.

Table 6: Actual Drucker Prager parameters

Parameter	**Value**
c	4.81 MPa
ϕ	54.6°
σ	4.37 MPa
α	0.246 MPa

When using the actual tested tensile and compressive strengths to estimate the c and ϕ for use in the non-linear Drucker Prager analysis, no equivalent plastic strain was detected. Hence, a vital crack that would cause failure is very unlikely.

It should be noted that the concrete is assumed to be homogeneous and have no internal cracks. Therefore, to predict where failure can be expected the following (weaker) material properties were used:

Table 7: Used Drucker Prager parameters

Parameter	**Value**
c	0.6 MPa
ϕ	47°
σ	0.65 MPa
α	0.225 MPa

Statistical data based on 146 shear tests carried out on samples from 10 different dams in the USA was used to estimate reasonable values for c and φ (Hansen, 1988). The samples had an average c- value of 0.6 MPa and φ-value of 47°. For this dam, these values are very conservative and take into account the possibility of internal cracks and defects in the concrete.

For the dynamic analysis the dynamic Young's modulus (E_d) was calculated using the following formula as recommended by Fulton's Concrete Technology (Owens, 2009:118):

$$E_c = 1.25E_d + 19\ GPa \tag{3.1}$$

Using this formula, a corresponding dynamic Young's modulus (E_d) of 37.6 GPa was calculated and used.

3.3.3 Geometry, mesh and boundary conditions

3.3.3.1 2D FEM analyses

The 2D FEM analyses have been carried out for three sections of the dam namely, at the central "river" section, left and right of the spillway. No section was taken in the NOC region since this part of the dam is usually more stable than the overspill section due to the additional weight of the NOC block. The positions of the sections analysed are presented below in Figure 24. The geometry of the sections that were analysed are presented in Figures 25 to 27. The dimensions of these sections were derived from construction drawings (see Appendix B).

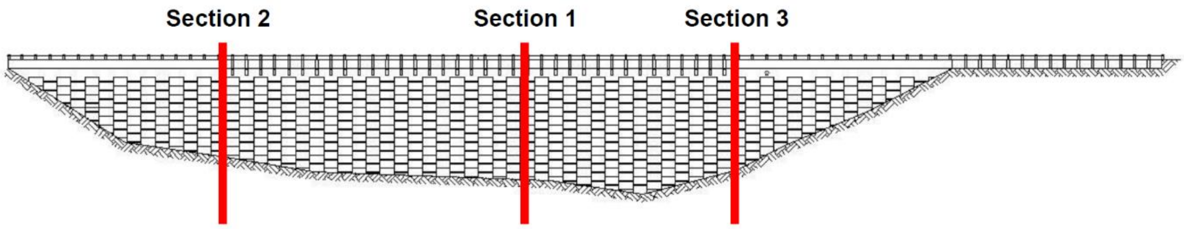

Figure 24: Downstream elevation of the dam showing the positions at which cross sections were analysed for the 2D analyses.

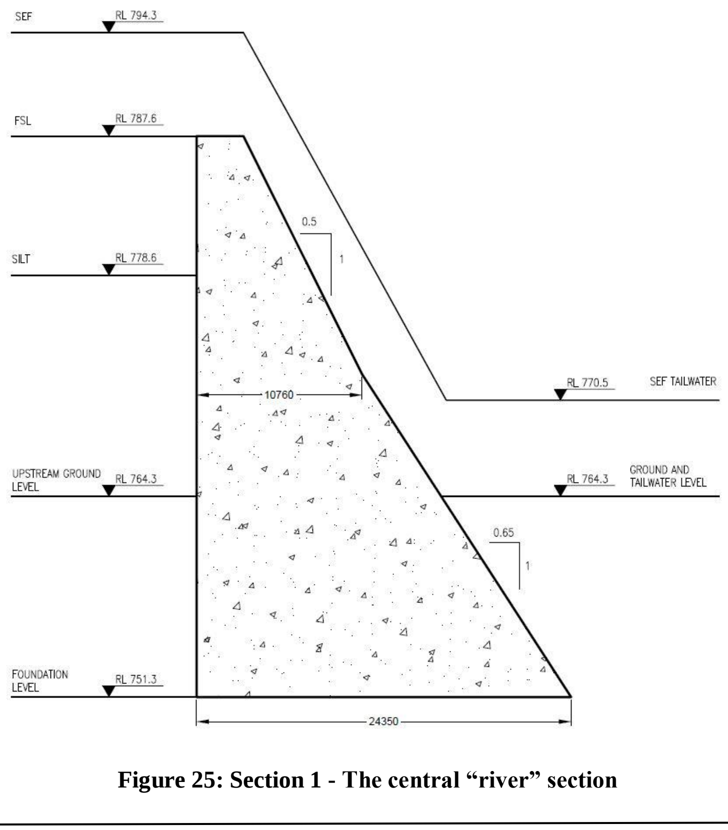

Figure 25: Section 1 - The central "river" section

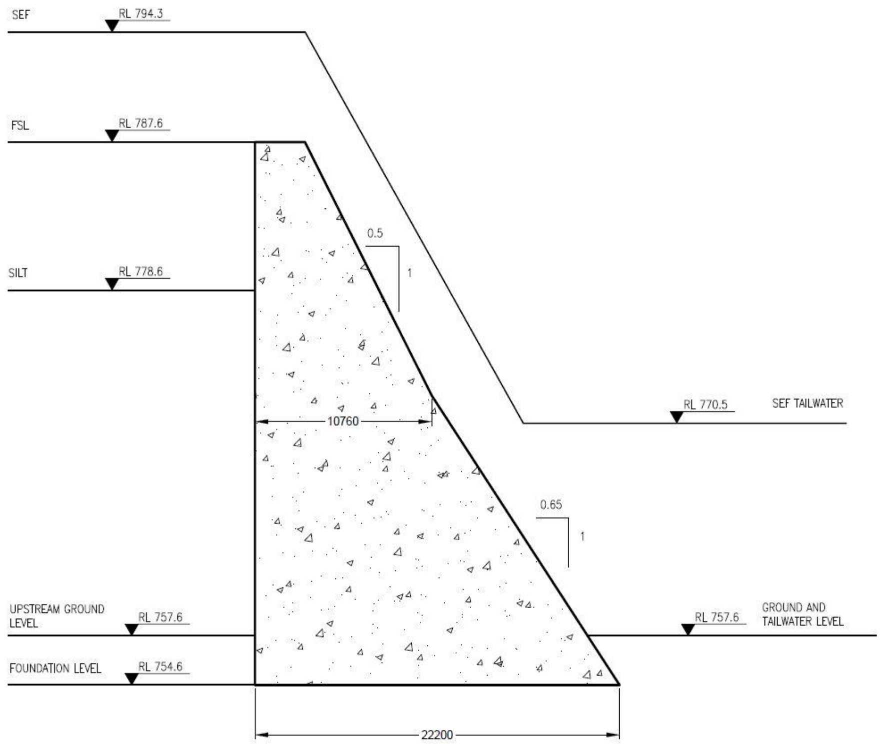

Figure 26: Section 2 – The section left of the spillway

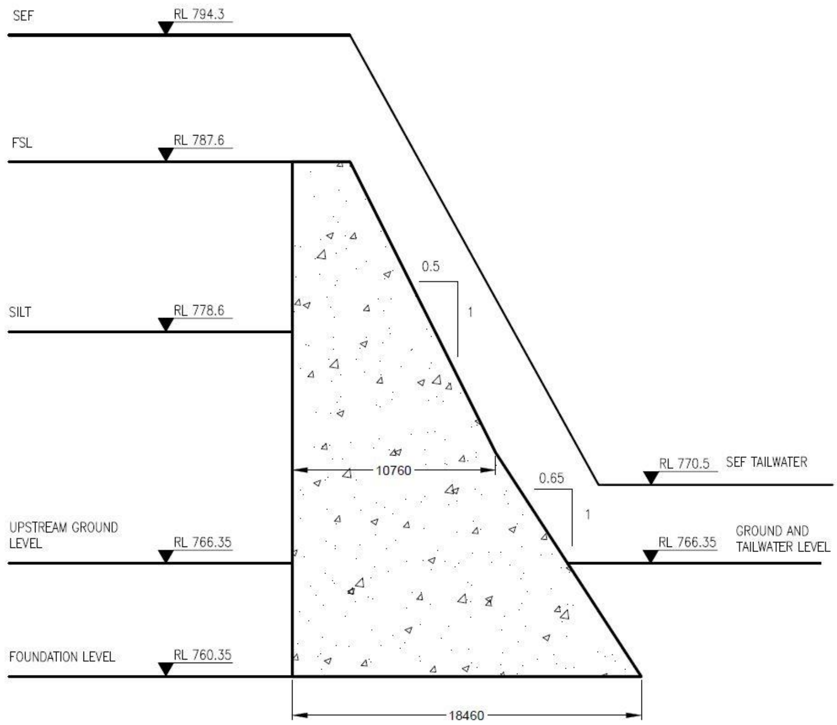

Figure 27: Section 3 - The section right of the spillway

A foundation block was added to model the foundation. For the 2D models the foundation block that was modelled extends approximately one dam height in the upstream, downstream and downward directions (as shown in Figure 28). Second Order 2D plain strain isoparametric (8-node Brick) elements were used for the dam wall and foundation as these elements are very flexible, save solving time and provide high-quality results.

2D boundary conditions

For the 2D FEM analyses the nodes on the bottom and side boundaries of the foundation block were all fixed for translation in the X and Y directions (As shown in Figure 28).

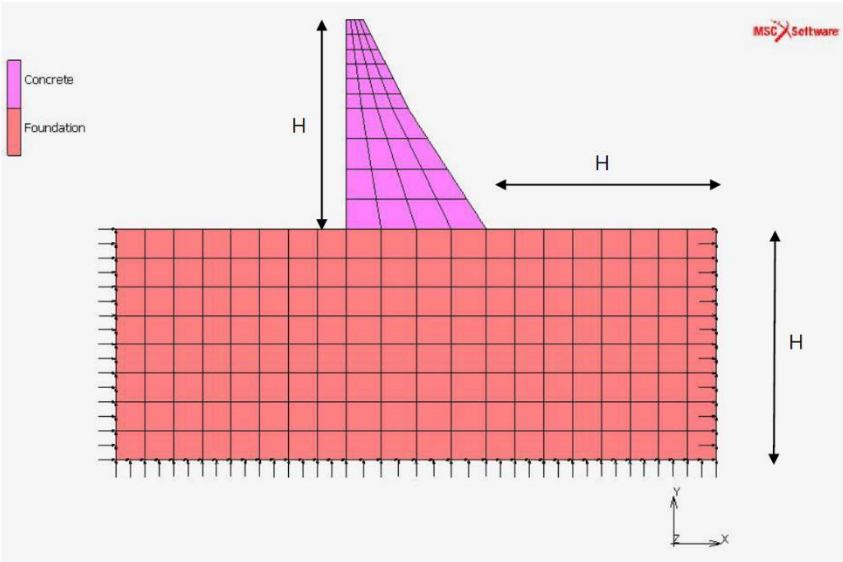

Figure 28: Example of the configuration of the 2D FEM

3.3.3.2 3D FEM analyses

The configuration of the dam and foundation block that was modelled and analysed for the 3D analyses is presented in Figure 29. The standard foundation block was added to the dam wall model to limit the singularity effect at the heel of the wall. The foundation block that was modelled also extends approximately one dam height in the upstream, downstream and downward directions (as shown in Figure 29). The dimensions of the dam wall were derived from drawings 130403/98, 6664/32 and 12520/92 (see Appendix B).

Second Order 3D Isoparametric (20-node Brick and 15-node Pentahedral) elements were used for the dam wall and foundation as these elements are very flexible, save solving time and provide high-quality results.

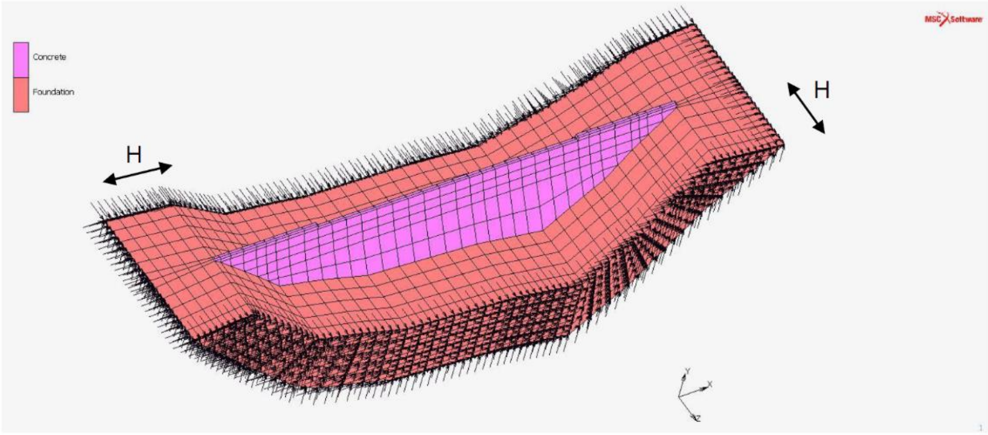

Figure 29: Configuration of the 3D FEM

3D boundary conditions

For the 3D FEM analyses the nodes on the bottom and side boundaries of the foundation block were all fixed for translation in the X, Y and Z directions (as shown above in Figure 29).

3.3.4 Loads and load combinations

The same static and dynamic load combinations were used for both the 2D and 3D FEM analyses of the dam.

3.3.4.1 Static load combinations

For the static (linear and non-linear Drucker Prager) analysis the following load conditions were examined:

Table 8: Static load combinations

Static Load Combinations	FSL	SEF	S	TW	DEP
Service	X		X	X	X
Extreme		X	X	X	X

Where:

FSL	=	Full Supply Level
SEF	=	Safety Evaluation Flood
S	=	Silt pressure
TW	=	Tailwater pressure
DEP	=	Downstream earth pressure

The FEM models were loaded using the following three-time steps:

- 1st Time step: Only gravity load, with an empty dam
- 2nd Time step: Dam filled to FSL
- 3rd Time step: Dam filled to SEF

Rankine's theory of lateral earth pressure was used to calculate the soil pressures acting on the dam wall. Both active and passive earth pressures were applied to the structure.

As a result of the uncertainty as to whether or not full uplift forces act on the dam wall, the FEM models were analysed with no uplift, 50% uplift and 100% uplift respectively (the uplift pressure distributions are shown in Figure 30).

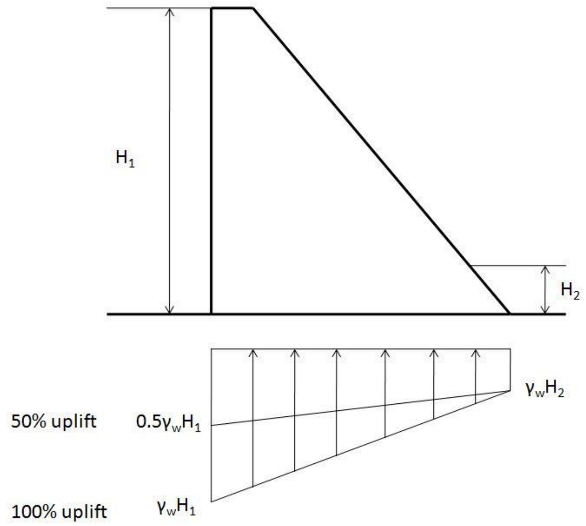

Figure 30: Diagram showing the uplift pressure distributions

3.3.3.2 Dynamic load conditions

Dynamic modal analyses

For the Dynamic Modal analyses the dynamic water pressure acting on the dam wall was modelled using Westergaard's theory of added mass (for 2D and 3D).

In the 2D analyses, the density of the first row of elements on the upstream face were modified according to Westergaard's added mass calculation. For the 3D analyses, the added mass was computed in the form of nodal masses in the horizontal direction and inserted at each node on the upstream face of the dam wall.

Response Spectrum Analysis

For the Response Spectrum Analyses the following load conditions were examined:

Table 9: Load combinations for the response spectrum analyses

Dynamic Load Combination	Added Mass	OBE	MCE
Abnormal: Operational Based Earthquake (OBE)	X	X	
Extreme: Maximum Credible Earthquake (MCE)	X		X

The Peak Ground Accelerations (PGA's) for both the OBE and MCE were obtained from the first DSE (Seddon, 1998:18) and confirmed by the seismic hazard map from SABS 0160:1989 (Rev. 1993) – see Figure 19. The following table presents the estimated PGA's for Nqweba Dam.

Table 10: Estimated PGAs for OBE and MCE recurrence intervals (Seddon et al., 1998:18)

Recurrence interval	Horizontal PGA
Operational Based Earthquake (OBE): 1 in 200 year	0.03 g
Maximum Credible Earthquake (MCE): 1 in 1 000 year	0.05 g

The well-known 1940 El Centro Earthquake is commonly used to approximate Response Spectrum loads in DWS as it provides conservative results. Figure 31 presents the 1940 El Centro Earthquake ground motion data. This Response Spectrum was scaled by using the estimated OBE (1: 200 year) and MCE (1:1000 year) Peak Ground Accelerations.

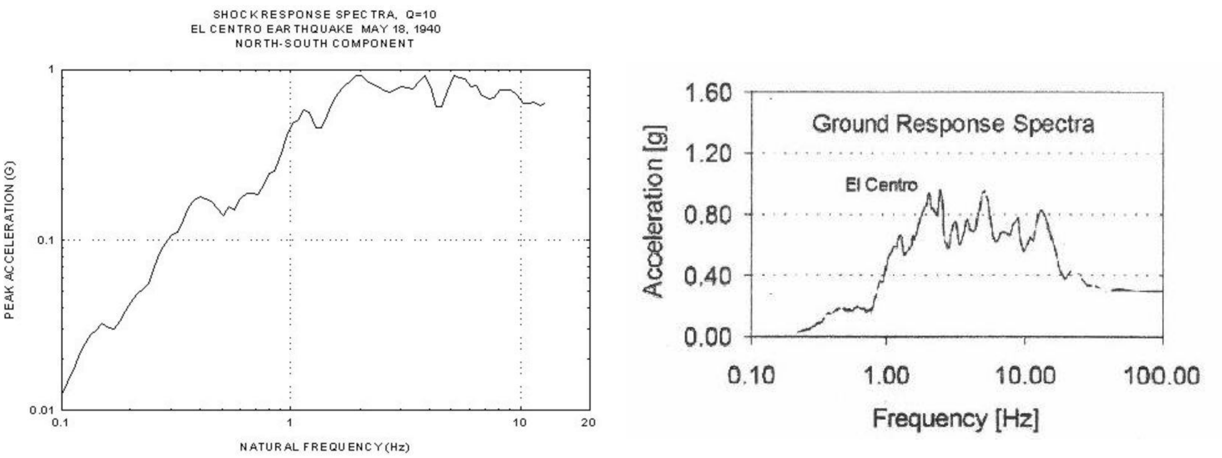

Figure 31: Ground response spectra of the 1940 El Centro Earthquake (El Centro Earthquake Page, n.d.)

As mentioned before, the profile of the El Centro 1940 Earthquake and the estimated PGA's were used to calculate the Response Spectrum for OBE and MCE recurrence intervals respectively. Table 11 presents the response spectrum used for the dynamic analysis.

Table 11: Response spectrum used for the dynamic analyses

Frequency (Hz)	Abnormal OBE: a (g)	Extreme MCE: a (g)
0.4	0.006	0.009
0.5	0.004	0.007
0.8	0.007	0.012
1	0.013	0.022
2	0.030	0.050
5	0.030	0.050
10	0.020	0.033
20	0.013	0.021
40	0.009	0.015
100	0.009	0.015

3.3.5 Results

3.3.5.1 Results of the 2D static FEM analyses

The results of the 2D static linear and non-linear analyses for no uplift, 50% uplift and 100% uplift (for all three sections analysed) are presented in Appendix D. Only the results for the worst-case scenario (i.e. for the full uplift assumption) of the section which displayed the highest stresses are presented in the table below. Visual representations of the wall displacement, normal stress as well as total equivalent plastic strain values for the SEF load case for section displaying the highest stresses are presented in Figure 32.

Table 12: Results of the 2D static linear and non-linear FEM analyses with full uplift assumed for the central section

Load Case and Analysis Type	Displacement (mm)	Maximum Principal Stress (MPa)	Minimum Principal Stress (MPa)
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.13	0.34	-0.93
"Non-linear" (Drucker Prager) Analysis	2.01	0.35	-0.86
Abnormal Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	4.44	0.82	-1.47
"Non-linear" (Drucker Prager) Analysis	4.50	0.51	-1.45

Note: Positive stress is tensile

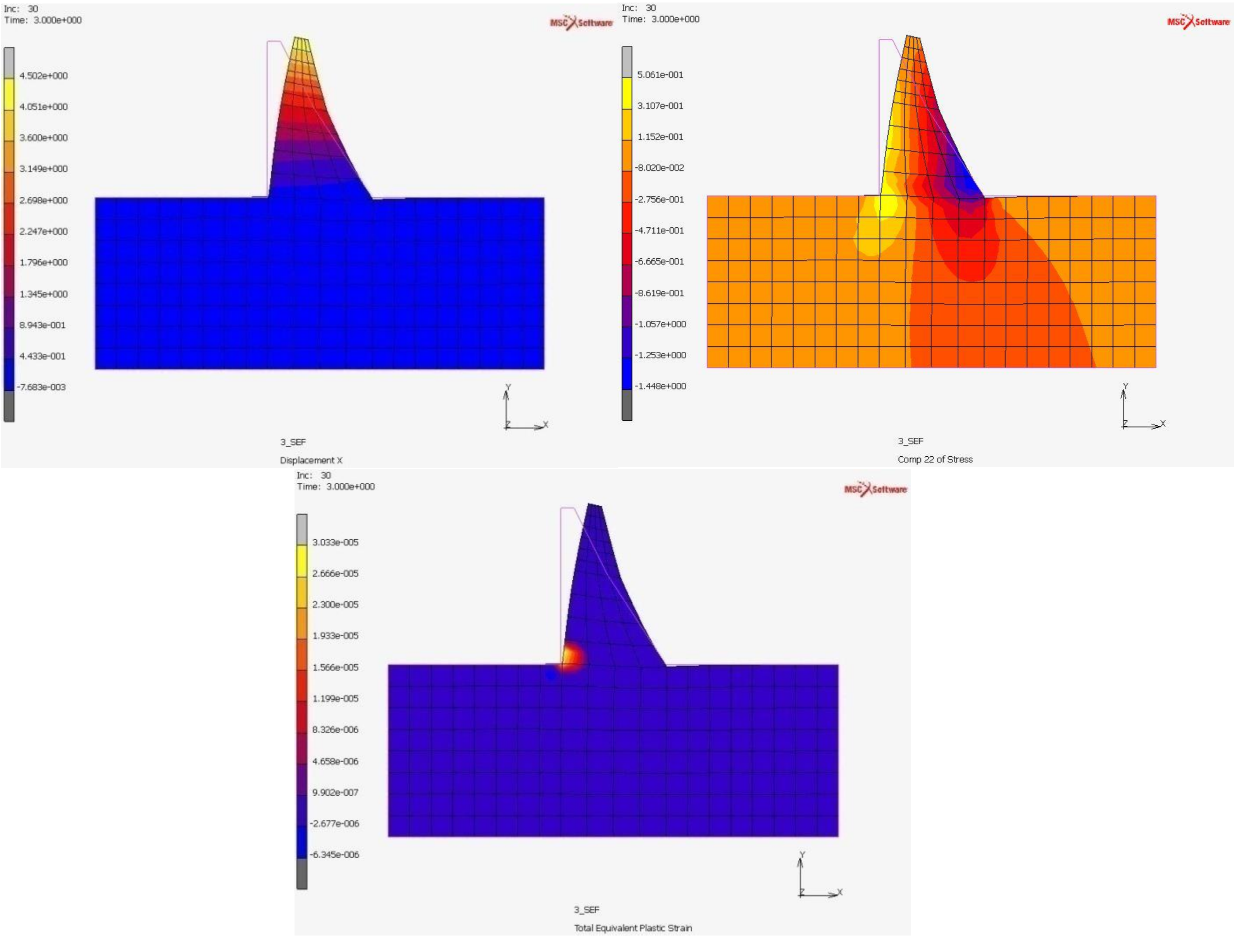

Figure 32: 2D FEM results - Displacement, normal stress and total equivalent plastic strain results for the SEF load case of the central section (full uplift assumed)

3.3.5.2 Results of the 3D static FEM analyses

The results of the 3D static linear and non-linear analyses for no uplift, 50% uplift and 100% uplift are presented in Appendix D. In the table below, only the results for the worst-case scenario (i.e. for the full uplift [100%] assumption) are presented. Visual representations of the wall displacement as well as minimum and maximum principal values of stress for the SEF load case are presented in Figures 35 to 37.

Table 13: Results of the 3D static linear and non-linear FEM analyses with full uplift assumed

Load Case and Analysis Type	Displacement (mm)	Maximum Principal Stress (MPa)	Minimum Principal Stress (MPa)
Service Load (FSL)	Downstream	S_1	S_3
Linear Elastic Analysis	2.35	0.58	-1.12
"Non-linear" (Drucker Prager) Analysis	2.35	0.58	-1.12
Abnormal Load (SEF)	Downstream	S_1	S_3
Linear Elastic Analysis	4.74	1.13	-1.88
"Non-linear" (Drucker Prager) Analysis	4.84	0.76	-1.92

Note: Positive stress is tensile

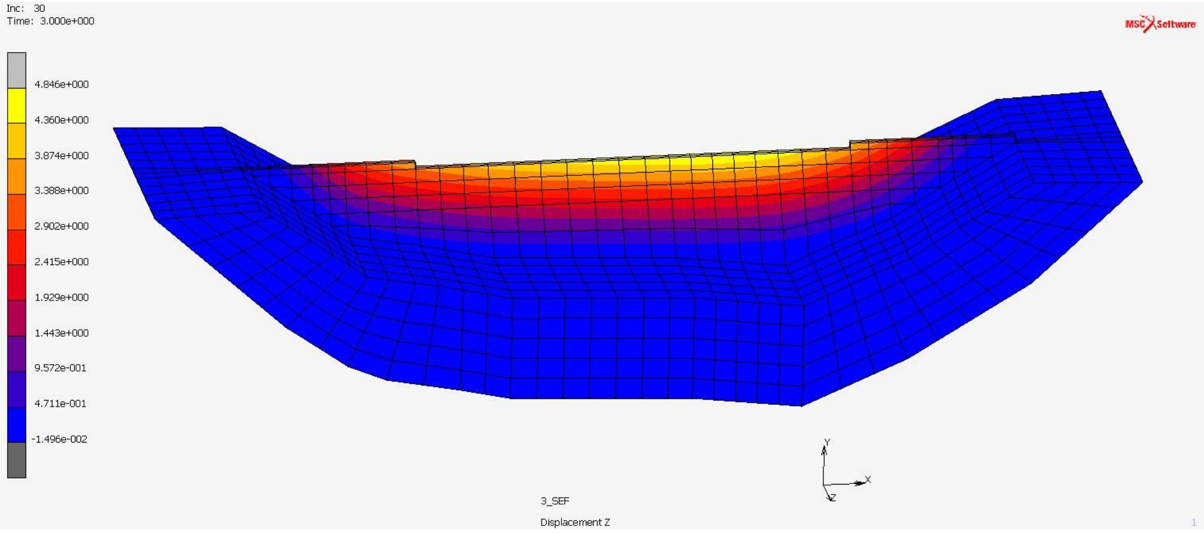

Figure 33: 3D FEM results - Displacement in the downstream direction (SEF condition, Full uplift assumed) in mm

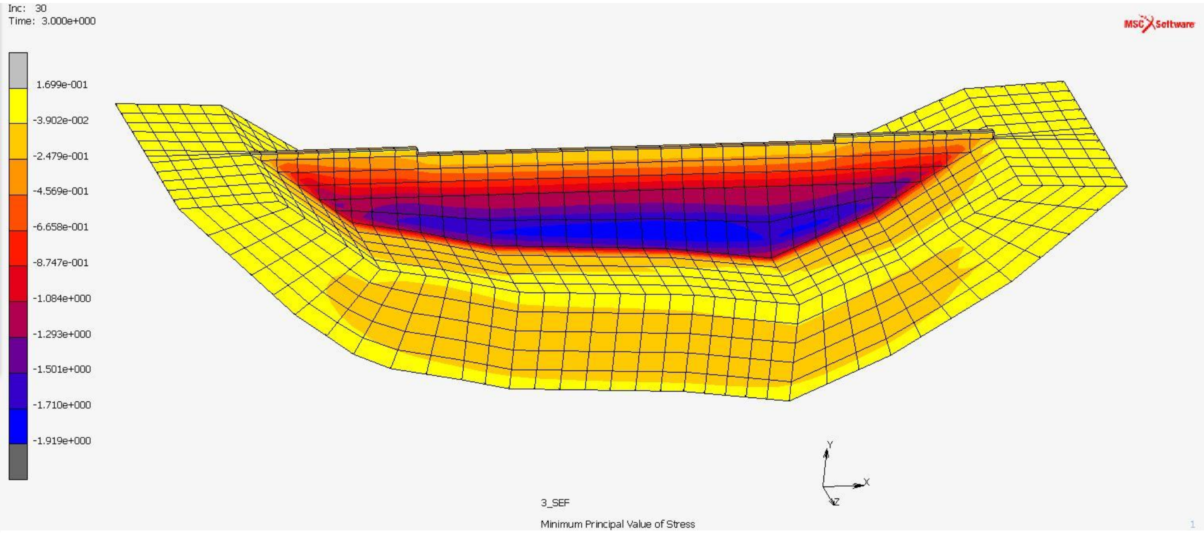

Figure 34: 3D FEM results - Minimum Principal Value of Stress (SEF condition, Full uplift assumed) in MPa

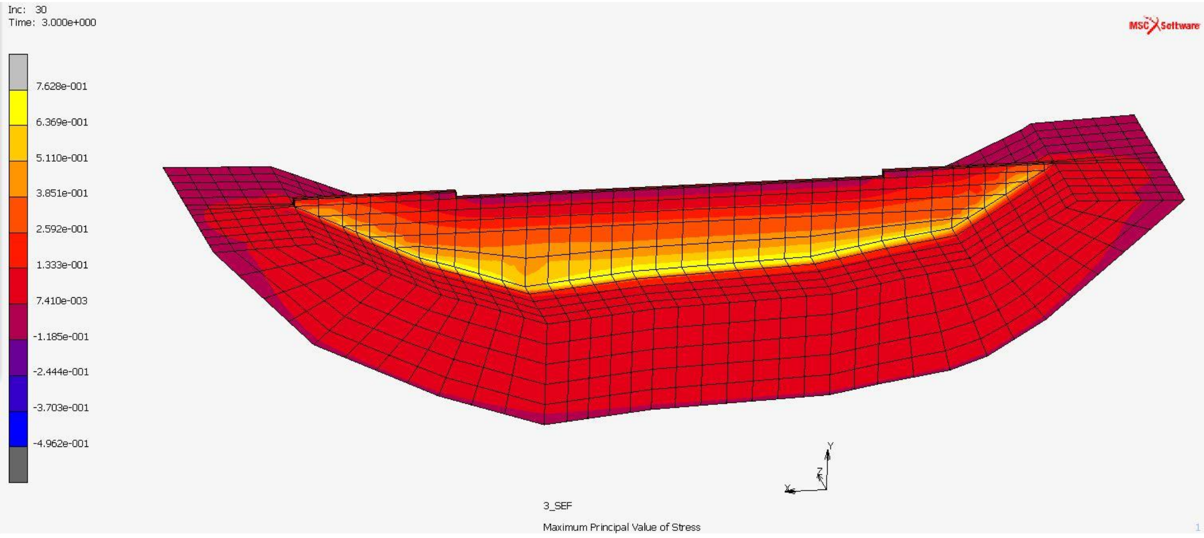

Figure 35: 3D FEM results - Maximum Principal Value of Stress (SEF condition, Full uplift assumed) in MPa

3.3.5.3 Results of the dynamic FEM analyses

As previously mentioned, the dam is located in an area of low seismicity – so significant damage to the structure during an earthquake is unlikely. The main purpose of the performing dynamic analyses is to determine the mode shapes and frequencies of the structure as this can be useful for long term structural health monitoring of the dam.

The first six mode shapes and natural frequencies for the dynamic modal analyses dam are presented in the Appendix E. The results of the dynamic response spectrum analyses are presented in Appendix D. These values are within the acceptable stress ranges for the abnormal and extreme load cases. They are also significantly lower than the results of the static analyses.

3.3.6 Discussion of results and the recommended way forward

3.3.6.1 Discussion of results

The results of the non-linear static analysis of the 2D analyses revealed that a maximum normal tensile stress (S_{22}) located at the heel of the dam of 0.345 MPa during FSL and 0.506 MPa during SEF conditions can be expected when considering full uplift. For the 3D analysis, a maximum principal tensile stress (S_1) located at the heel of 0.578 MPa during FSL and 0.763 MPa during SEF conditions can be expected when assuming full uplift forces. The tested tensile strength of the concrete is approximately 3 MPa. The results also indicate that the maximum tensile stress, is generally localised at the heel of the wall.

The table below compares the results of the 2D analysis of Nqweba dam to that of other concrete gravity dams such as Flag Boshielo (Dureiux, 2010) and De Hoop Dam (Dureiux, 2007).

Table 14: Comparison of 2D FEM analyses results to other concrete gravity dams

Load Condition	FSL	SEF
Dam	**Normal stress at heel, S_{22} (MPa)**	**Normal stress at heel, S_{22} (MPa)**
Nqweba Dam (Non-linear static)	0.345	0.506
Flag Boshielo Dam (Non-linear static)	0.200	0.550
De Hoop Dam (Non-linear static)	1.310	1.430

Note: Positive stress is tensile

The results of the dynamic analyses indicate that no severe structural damage would occur during an Operational Based Earthquake (OBE) or Maximum Credible Earthquake (MCE) event. The stresses induced by the OBE and MCE are very low therefore no severe structural damage is expected, and the structure would be stable during such events. The first six mode shapes of the dam (for the 3D model) ranges between 7.23 and 12.26 Hz. It should however be noted that no calibration was done to confirm these modes of vibration.

The results of the FEM analyses performed suggests that the dam fulfils the failure evaluation criteria for gravity dams and that it is safe for high flood levels such as the SEF condition. The development of a vital crack that would cause failure of the dam is considered to be very unlikely.

3.3.6.2 Recommendations based on the results of the FEM analyses

The following recommendations are made based on the results of the FEM analyses:

- Install new piezometers (and/or restore the existing piezometers) to confirm pore pressures and uplift forces acting on the dam wall
- Perform Ambient Vibration Testing (AVT) to confirm the natural frequencies and mode shapes of the dam, calibrate the FEM model, and also monitor the stiffness of the structure to detect any damage (i.e. cracks) or weakening of the structure over time
- Perform shear box tests on cores (extracted from the concrete) to determine the actual cohesion and angle of friction for the dam
- No major structural rehabilitation, besides refurbishing the parapet walls (as recommended in previous DSE's) to safeguard the dam is deemed necessary at this stage.
- Monitoring of the structure should continue

3.3.6.3 The recommended way forward in accordance with South African dam safety legislation

It should be noted that, although the findings are believed to be very useful, this study is not an official Dam Safety Evaluation (DSE). The last official DSE in terms of the Dam Safety Regulations was done in 2008. As discussed in Section 2.8, the Dam Safety Regulations states that for the evaluation of the safety of a Category III dam, an APP assisted by a professional team is required. Furthermore, in the case of a dispute, it is required that an independent expert or team of experts be appointed to evaluate the findings.

Therefore, in accordance with these legislative requirements it is recommended that an independent expert or team of experts be appointed to evaluate the findings and recommendations of this study and perform the next DSE. In the opinion of the author, ultimately, the APP and professional team (or experts) approved by the relevant authorities in terms of the Dam Safety Regulations are responsible for making final recommendations and decisions with regard to the safety of the dam.

3.4 CHAPTER CLOSURE

In this chapter, an evaluation of the structural safety of Nqweba dam was performed as a case study. The next chapter presents and discusses the proposed framework for the evaluation of the structural safety of existing concrete gravity dams that was developed based on the research review and findings of the case study.

CHAPTER 4 – THE PROPOSED FRAMEWORK FOR THE EVALUATION OF THE STRUCTURAL SAFETY OF EXISTING CONCRETE GRAVITY DAMS

4.1 THE PROPOSED FRAMEWORK

In the literature review, a concise background of gravity dams, their potential failure modes, some historical failure incidents, an insight to common structural safety evaluation methods, as well as some legislative requirements for the evaluation of the safety of large dams in South Africa was presented.

Thereafter, in Chapter 3, an evaluation of the structural safety of Nqweba dam was performed as a case study. In the case study, a more advanced type of structural analysis was done, and the results suggested that major structural rehabilitation is not necessary. Based on the research done in the literature review and the case study, a proposed framework for the evaluation of the structural safety of existing concrete dams was developed. This a summary of this recommended framework is presented in Figure 36.

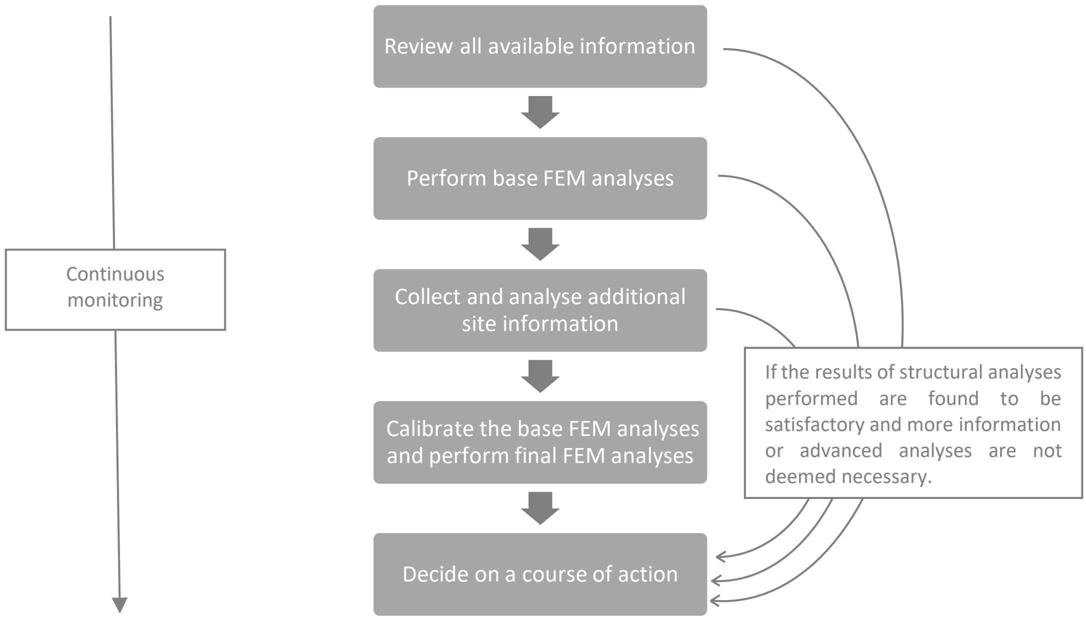

Figure 36: Summary of the recommended framework for the structural evaluation of existing concrete gravity dams

The successive stages of the framework (as presented in Figure 36) are discussed in the forthcoming sections of this chapter.

It is recommended that a deterministic approach be followed up until the last step of the framework. In this final step probabilistic approaches are deemed useful and complementary to the prior deterministic approaches. The probabilistic analyses can then be used to assist decision makers.

4.2 REVIEW ALL AVAILABLE INFORMATION AND STUDIES

As an initial step, all of the available information and studies should be critically reviewed. This information may include but is not limited to existing drawings, previous Dam Safety Evaluations (DSE's), engineering geology reports, existing instrumentation and monitoring results and so forth.

In this phase the site-specific conditions as well as issues which are of concern should be identified. It is also useful to review the results of all prior studies that have been carried out for the structure.

Existing instrumentation and monitoring information can provide a useful insight into the behaviour and performance of the structure over time. For example, it can provide information about how the structure performs during high or low water levels, or even during a high flood event.

It is assumed that, for an existing structure, a simple (quick check, "Classical Method") stability analyses would have previous been done. This stability analyses should also be reviewed. If the stability analyses find that the structure is safe, then no further, advanced analyses are necessary. If not, then a more advanced kind of analysis (a FEM analysis) should be done. In the authors opinion, the Classical Method of analysis should only be used as a "quick check", and not as a basis for recommendations of rehabilitation work. Therefore, it is not advised to make final decisions to rehabilitate an existing concrete gravity dam purely based on results of a basic form of stability analysis such as the Classical Method. As previously mentioned, it is emphasised that if the structure fails to comply with the stability criteria of the Classical Method, then the FEM should be used to verify the results of the Classical Method.

4.3 PERFORM BASE FEM ANALYSES

When simple stability analysis methods (such as the Classical Method) suggest that the structure is "unsafe", advanced methods such as FEM analysis should be done. Initial (or base) FEM analyses should then be performed using the available information and assumptions. If enough site information (monitoring results, material test results, etc.) is available, then the initial FEM analyses can be calibrated, and final course of action determined. If not, then additional site information should be collected and analysed. It is recommended that the FEM analysis is documented and checked by an experienced engineer.

If the initial FEM results are acceptable then this is an indication that the integrity of the structure is adequate. Nevertheless, it is recommended that more information should be collected and analysed, and the FEM should be calibrated to verify the results.

4.4 COLLECT AND ANALYSE ADDITIONAL SITE INFORMATION

If deemed necessary in the previous step, additional site information should be collected and analysed. The purpose of collecting additional site information would be to answer relevant site-specific questions which may have surfaced as well as to calibrate and refine the FEM analyses to verify the results obtained. This information may be collected by means of additional instrumentation and monitoring or tests and investigations.

4.5 CALIBRATE THE BASE FEM ANALYSES AND PERFORM FINAL FEM ANALYSES

Once the additional information has been collected and analysed, the FEM analyses should be calibrated. The calibration of the FEM analyses will ensure that the model corresponds to real life, site-specific conditions and actual behaviour of the structure. It also allows for the results to be verified.

As mentioned in Section 2.6.3.3 of Chapter 2, the static model can be calibrated by altering its material properties (i.e. manipulating its stiffness) until the FEM displacements approximately coincides with the actual measured displacements from dam monitoring readings (e.g geodetic surveys). Calibration of the dynamic FEM can be achieved by adjusting the "added mass" and comparing the calculated natural frequency modes to the actual tested ambient vibration modes.

4.6 DECIDE ON COURSE OF ACTION

This step may, in some instances, be taken at an earlier stage. For example, if the results of structural analyses performed are found to be satisfactory and more information or advanced analyses are not deemed necessary.

If deemed necessary (based on the analyses performed), there are various alternatives for the improvement of the safety of the structure. These alternatives range from doing nothing to the existing structure and only implementing a comprehensive warning system to major rehabilitation, or even decommissioning the structure. The alternatives for improvement should then be investigated.

As previously mentioned in Section 2.6.4 of the literature review, probabilistic methods (or risk-based analyses), are complimentary and not contrary to deterministic methods of analysis (Oosthuizen et al.,1991:144). They can be performed to confirm the results of deterministic analyses. This kind of analysis can also be done to assist decision makers in obtaining an optimum solution (most suitable alternative) and prioritising rehabilitation works.

In the opinion of the author, ongoing monitoring is extremely important – especially for existing, ageing structures. As long as there are issues of concern, it is important to continue with the monitoring of the structure as important insights to its behaviour and performance may be revealed by doing so. Appropriate decisions can then be made, and analyses can be done based on actual measured data.

As required by dam safety legislation in the country, for this final step of the framework, the APP and professional team (or experts) approved by the relevant authorities in terms of the Dam Safety Regulations are responsible for making final recommendations and decisions with regard to the safety of the dam.

4.7 CHAPTER CLOSURE

This chapter presented and discussed the proposed framework for the evaluation of the structural safety of existing concrete gravity dams that was developed based on the research review and findings of the case study. The next chapter concludes this dissertation and some recommendations for future research are presented.

CHAPTER 5 – CONCLUSION AND RECOMMENDATIONS

5.1 SUMMARY

The focus of this research study was specifically on the evaluation of the structural safety of large concrete gravity dams. Chapter 2 presented a concise background of concrete gravity dams, their potential failure modes and some historical failure incidents, an insight to common structural safety evaluation methods, as well as some legislative requirements for the evaluation of the safety of large dams in South Africa. In Chapter 3, an existing large concrete gravity dam was evaluated as a case study. The findings of previous dam safety evaluation reports for the dam led to it being labelled as essentially “unsafe” in the case of the occurrence of a large flood. Typically, this would mean that the dam must be rehabilitated to improve its safety. However, it was demonstrated that with the use of modern structural analysis methods and the consideration of all the available information, a more realistic structural safety evaluation can be achieved. In turn, more economical optimizations in terms of rehabilitation can be accomplished and costs of unnecessary, expensive rehabilitation can be avoided.

Based on the research and analyses done in the literature review and the case study, a proposed framework for the evaluation of the structural safety of existing concrete dams was developed. This framework was discussed and presented in Chapter 4. In summary, for existing large concrete gravity dams, the recommended steps to be taken are; review all available information, perform base FEM analyses, collect and analyse additional site information, calibrate the base FEM analyses and perform final FEM analyses, and decide on a course of action. It was also emphasised that the structural analyses should consider site-specific information. Using conservative assumptions and basing decisions on outdated or simple kinds of analyses can result in unnecessary and expensive recommendations.

Some concluding remarks and recommendations for future studies based on this dissertation are presented in this chapter.

5.2 CONCLUDING REMARKS

The following concluding remarks are made based on this research study for the evaluation of the structural safety of existing concrete gravity dams:

- FEM analyses should be done before making final decisions to rehabilitate an existing structure. The decision to rehabilitate shouldn’t be made based on the simple Classical Method of analysis. Compared to the past, it is much cheaper and quicker these days to perform advanced structural analyses (such as FEM analyses) – thus we should make good use of it.

- Comprehensive instrumentation and monitoring systems are important as they provide a better understanding of site-specific conditions and the actual behaviour of the existing structure, compared to making conservative assumptions.
- The framework developed in this dissertation can be a useful guide for future safety evaluations of existing concrete gravity dams.
- The potential benefits of using this recommended framework includes avoiding unnecessary rehabilitation work, as well as significant time and cost savings.

5.3 RECOMMENDATIONS FOR FURTHER RESEARCH

Based on the research done in this study the following recommendations are suggested for future studies:

- Quantify possible savings that could be made by performing advanced structural analyses and collecting and analysing site-specific information, compared to hastily choosing the conservative option of rehabilitation
- The accurate calibration of FEM models of existing concrete gravity dams
- The development of a framework for the evaluation of the structural safety of other types of dams

CHAPTER 6 - REFERENCES

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APPENDIX A: CLASSIFICATION OF DAMS

Annexure

Tables for the classification of dams with a safety risk

Table 1: Size classification

Size class	Maximum wall height in metres (m)
Small	Less than 12 m.
Medium	Equal to or more than 12 m but less than 30 m.
Large	Equal to or more than 30 m.

Table 1 must be read together with subregulation 2(2).

Table 2: Hazard potential classification

Hazard potential rating	Potential loss of life	Potential economic loss	Potential adverse impact on resource quality
Low	None	Minimal	Low
Significant	Not more than ten ...	Significant	Significant
High	More than ten	Great	Severe

Table 2 must be read together with subregulation 2(3).

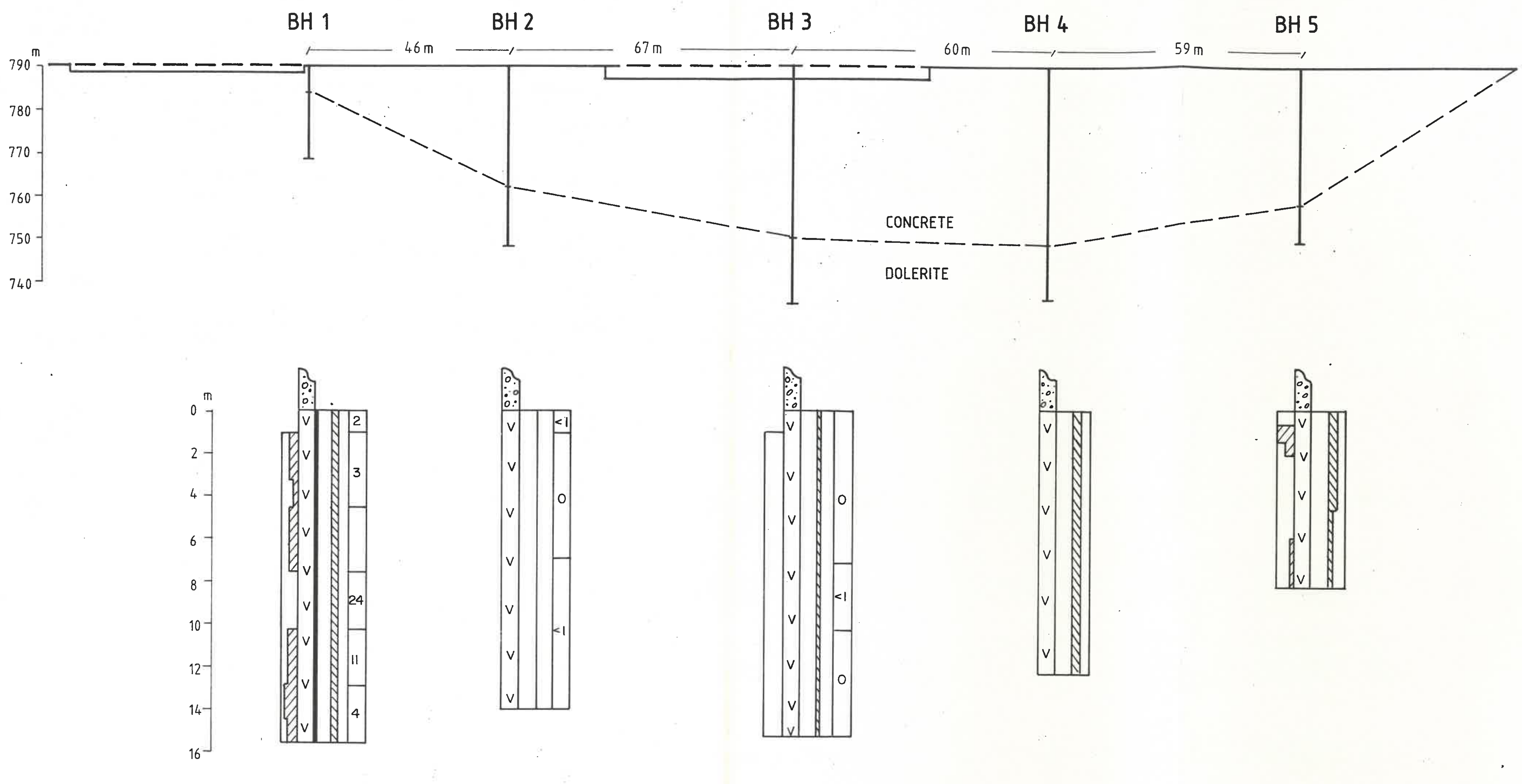
Table 3: Category classification of dams with a safety risk

Size class	Hazard potential rating		
	Low	Significant	High
Small	Category I	Category II	Category II
Medium	Category II	Category II	Category III
Large	Category III	Category III	Category III

Table 3 must be read together with subregulation 2(4).

APPENDIX B: DRAWINGS

G. P.-S (L).



R. Q. D. :

0 - 25 %
26 - 50 %
51 - 75 %
76 - 90 %
> 90 %

MATERIAL :

Concrete
Dolerite

Slight
Not W.

Weathering

JOINTING :

Medium
Wide
Very wide

3

Water Loss, Lugeons

GEOLOGIESE OPNAME
GEOLOGICAL SURVEY

Privaatsak
Private Bag X112
PRETORIA
0001

AFDELING INGENIEURSGEOLOGIE
ENGINEERING GEOLOGY DIVISON

SEKSIE
SECTION

DAM AND UNDERGROUND INVESTIGATIONS

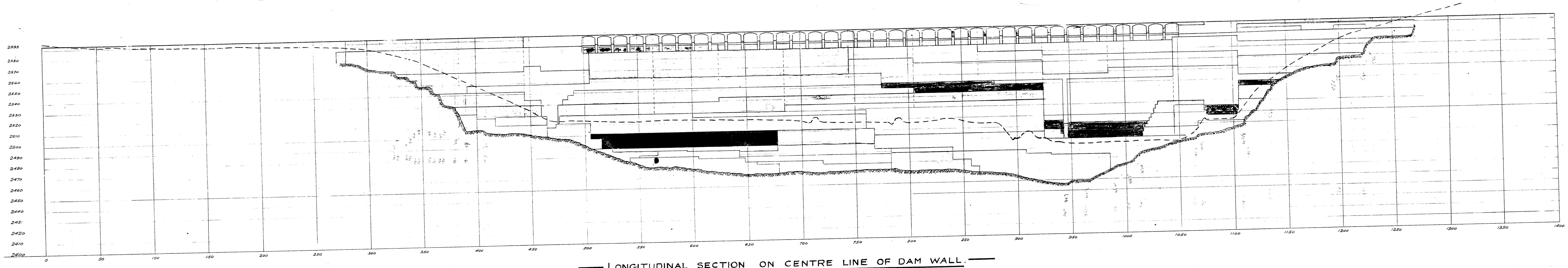
SUNDAYS RIVER SCHEME : VAN RYNEVELDSPASS DAM, GRAAFF-REINET

FIG. 1 : POSITIONS AND GEOLOGICAL COLUMNAR SECTIONS OF BOREHOLES ON DAM WALL.

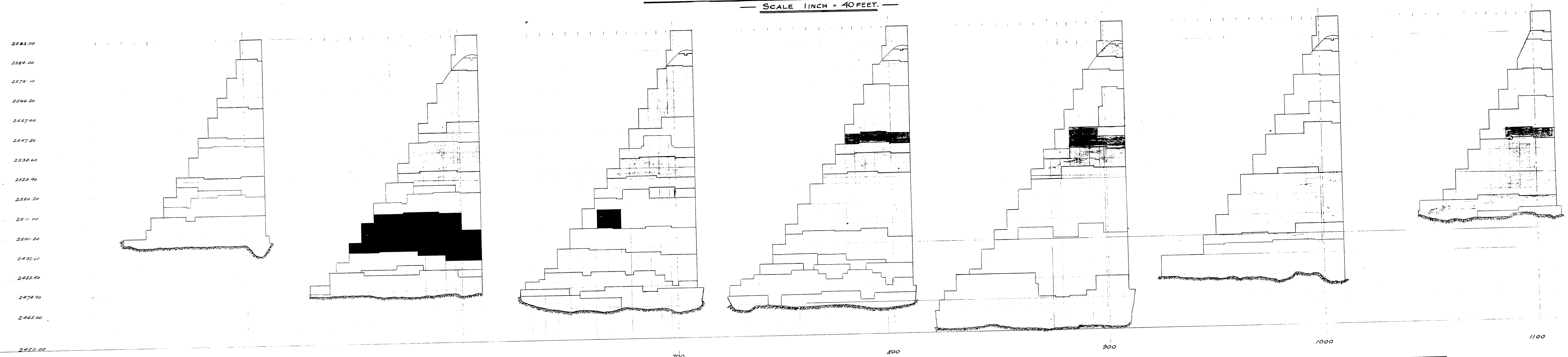
SAAMGESTEL DEUR / COMPILED BY	A. SCHALL	DRAWN / GETEKEN	V. PEER	KLASSIFIKASIE/CLASSIFICATION
TEGNIESE KONTROLE / TECHNICAL CONTROL		DATUM / DATE	JULY 1988	PLAN No./Nr
VERWYSINGS / REFERENCES		KOPIEREG	COPYRIGHT	

MD 1138

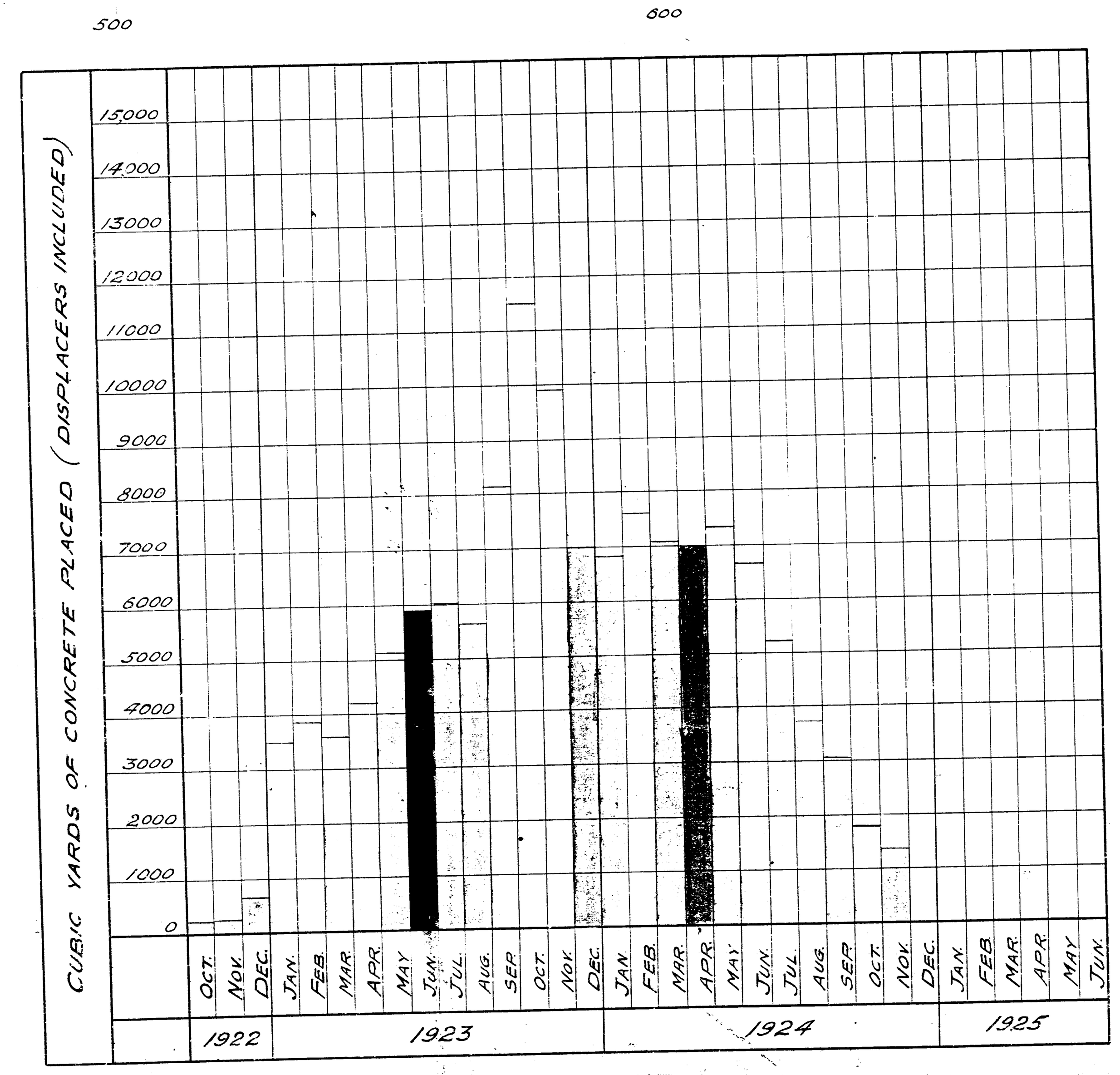
PROGRESS OF CONSTRUCTION



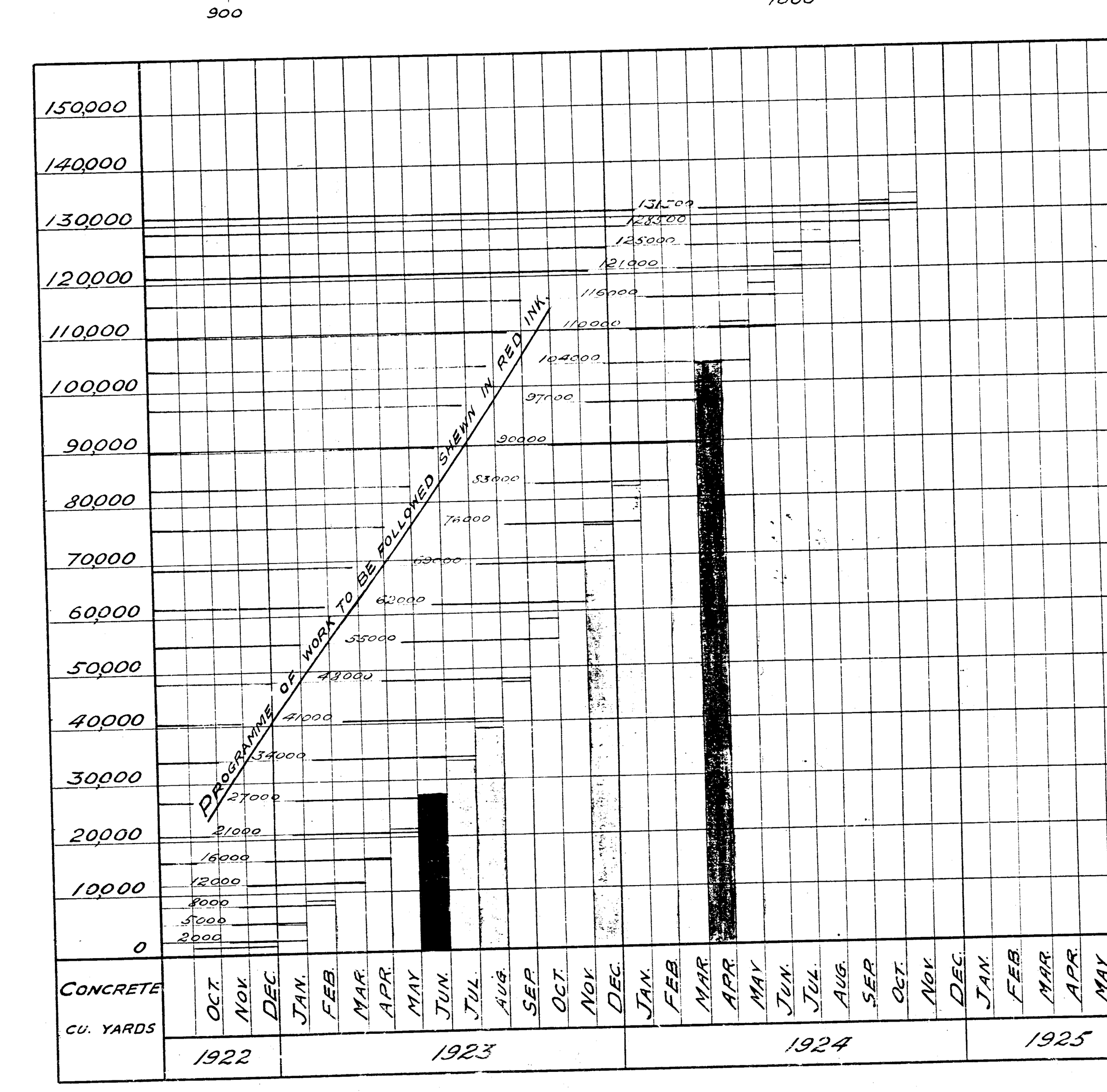
— LONGITUDINAL SECTION ON CENTRE LINE OF DAM WALL —
— SCALE 1 INCH = 40 FEET —



— CROSS SECTIONS OF DAM WALL —
— SCALE 1 INCH = 20 FEET —



—MONTHLY CONCRETE PLACED IN MAIN WALL—

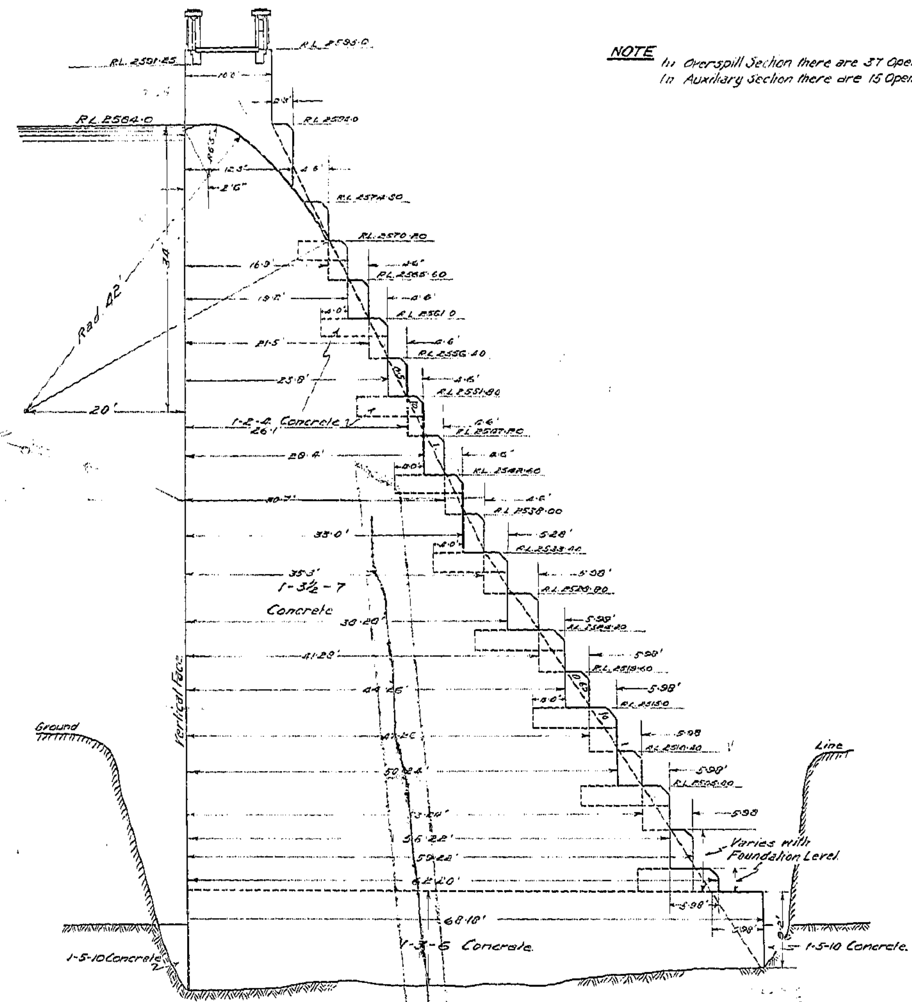


— TOTAL CONCRETE IN WALL TO DATE —

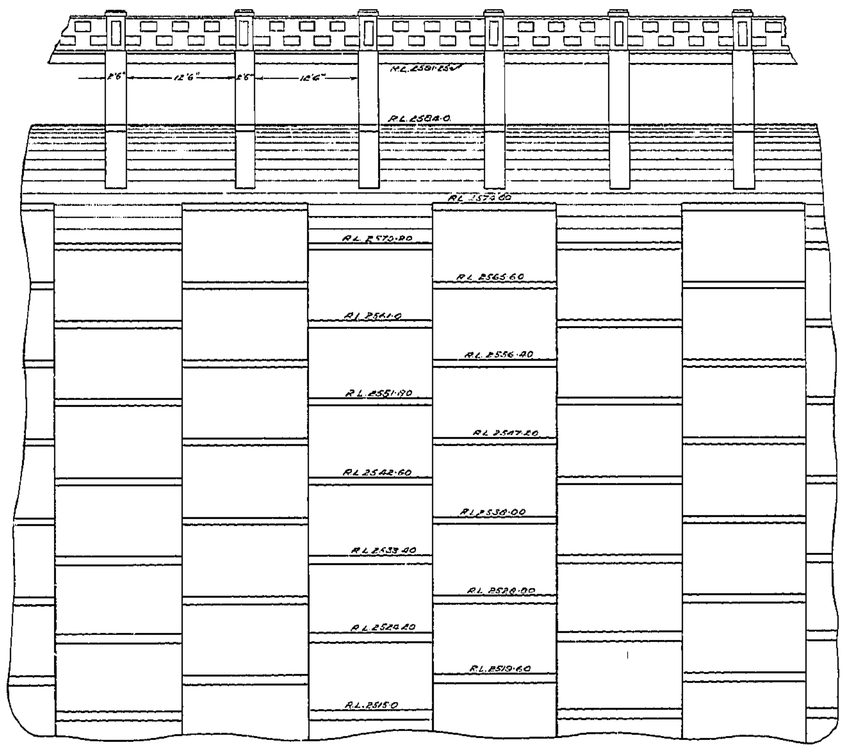
VAN RYNEVELDS PASS IRRIGATION SCHEME

REG. No: 130403/98

NOTE In Overspill Section there are 37 Openings.
In Auxiliary Section there are 15 Openings.



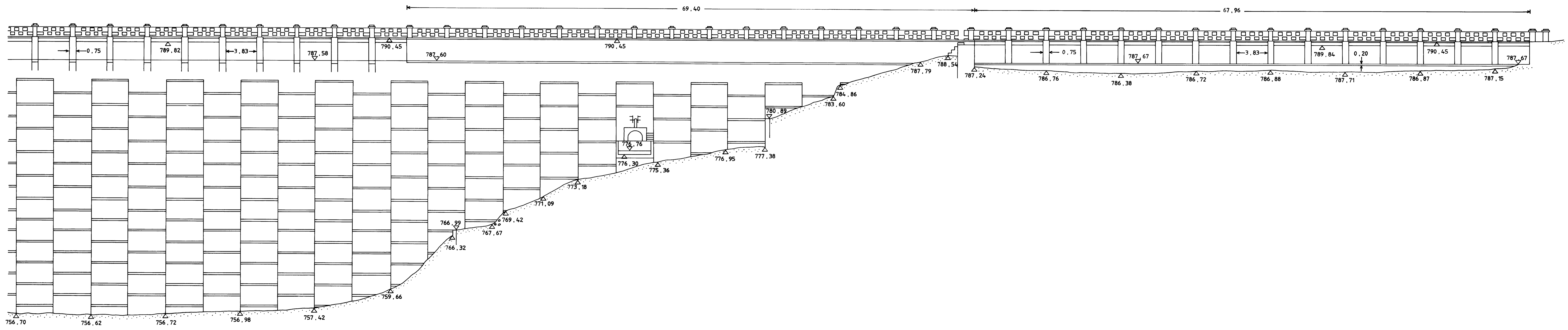
TYPE OVERSPILL SECTION.
SCALE: 1 INCH = 10 FEET.



PART ELEVATION OF DOWNSTREAM FACE.
SCALE: 1 INCH = 10 FEET.

D.A.M.

CAPE.	IRRIGATION DEPARTMENT		GRAAFF REINET.
VANRYNEVELDS PASS IRRIGATION SCHEME.			
DETAILS OF DAM (AS CONSTRUCTED.)			
Drawn by. Traced by. BXS. 21-1-32. Checked by.	HEAD OFFICE SUPERINTENDING ENGINEER	WATER COURT Nº 8. A.D. LEWIS. DIRECTOR OF IRRIGATION.	DRAINAGE AREA. XIII - 1. SUNDAYS RIV. REG. Nº 6664/32.



KOÖRDINAATSTELSEL LO. 25° H.H. VANAF METERS BO SEESPIEEL			
STASIE	Y	X	H.H.

CO-ORDINATE SYSTEM LO. 25° R.L. FROM METRES ABOVE M.S.L.			
STATION	Y	X	R.L.

VELUITLEG / SHEET LAYOUT

HOR. 1: 250
VER. 1: 250

m

SKAAL/SCALE

DIREKTORAAT OPMETINGSDIENSTE — **DIRECTORATE SURVEY SERVICES**

PRIVAATSAK X313 PRIVATE BAG
PRETORIA
0001

OPGEMEET SURVEYED	H. LODEWYK	05/92	NAGETREK TRACED M. MATTHEE	06/92
SAAMGESTEL COMPILED	A.T. KOLLMANN	06/92	TOPOGRAFIE NAGESIEN TOPOGRAPHY CHECKED	
FOTOGRAFIE PHOTOGRAPHY			KADASTRAAL NAGESIEN CADASTRAL CHECKED	

Nr./No.	HERSIENING/REVISION		DATUM DATE

REPUBLIEK VAN SUID-AFRIKA
DEPARTEMENT VAN WATERWESE

REPUBLIC OF SOUTH AFRICA
DEPARTMENT OF WATER AFFAIRS

VAN RYNEVELDSPASDAM

VOORAANSIG : STROOMAF VAN LINKERFLANK DAMWAL

VAN RYNEVELDSPAS DAM

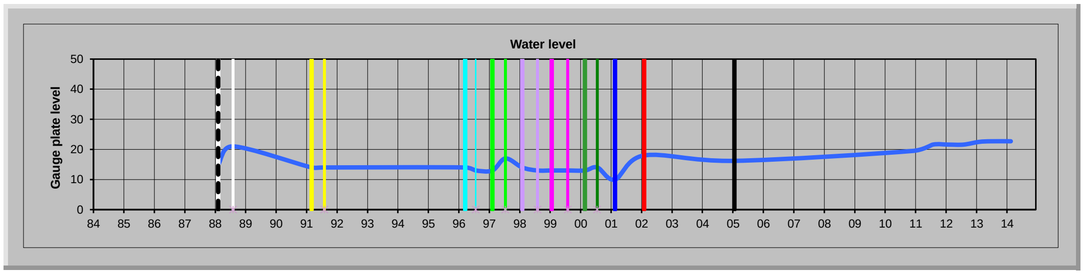
FRONT ELEVATION : DOWNSTREAM OF LEFT FLANK DAM WALL

DISTRIK DISTRICT	GRAAFF-REINET	VEL SHEET 1 VAN 1	LOK.NR./LOC.NO. N 120/012	REG.NR./REG.NO. 112 520/92
OPMETINGSBOEKE SURVEY BOOKS	KODE CODE	ASB | DAU | ELE | |		

EC 11056/92

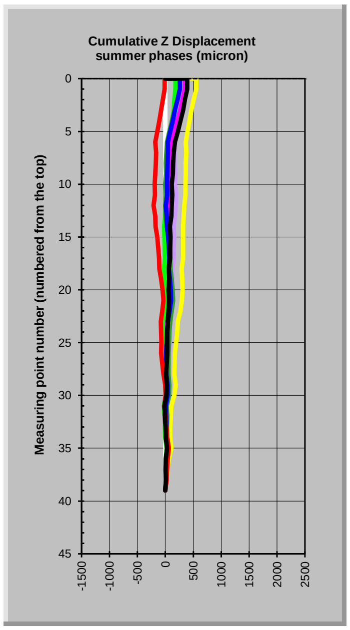
APPENDIX C: SLIDING MICROMETER RESULTS

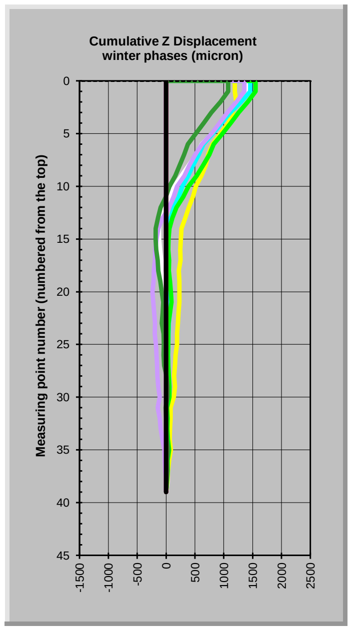
Vrvm1 1988 - 2005

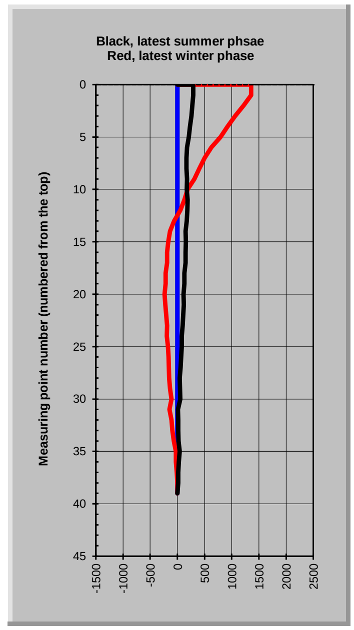


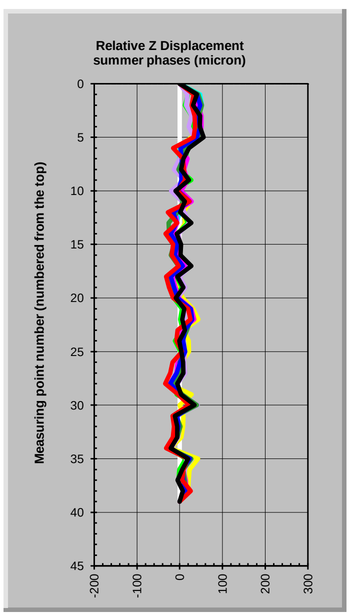
Legend: 02/88, 03/91, 03/96, 02/97, 02/98, 01/99, 03/00, 03/01, 02/02, 02/05

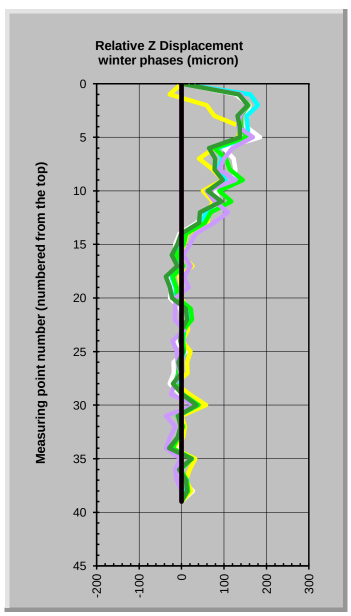
Rock/concrete contack   Base date

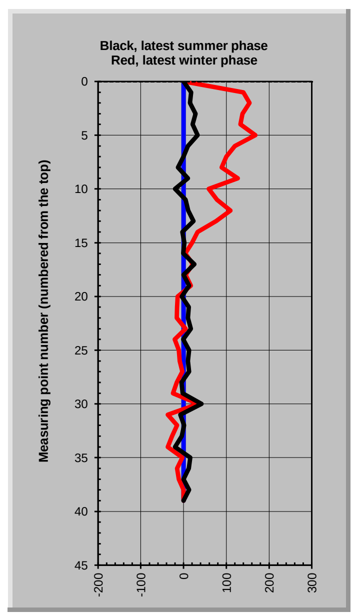






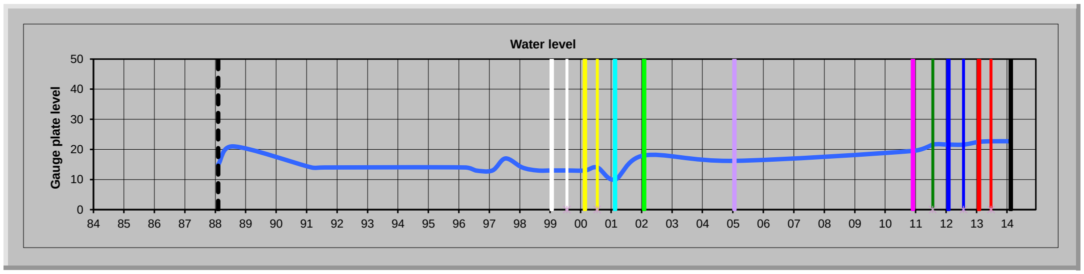






Positive is shortening

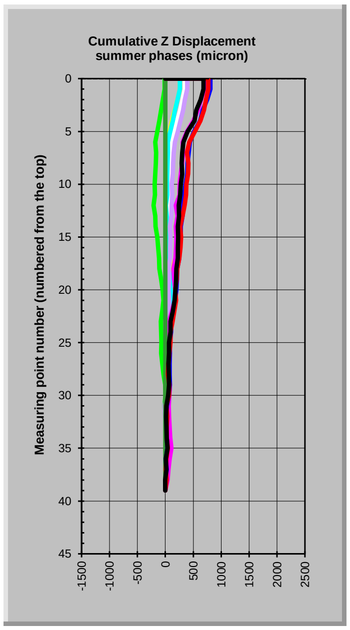
Vrvm 1 1999 - 2014

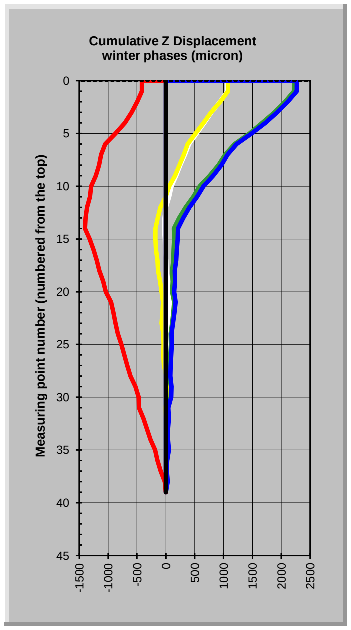


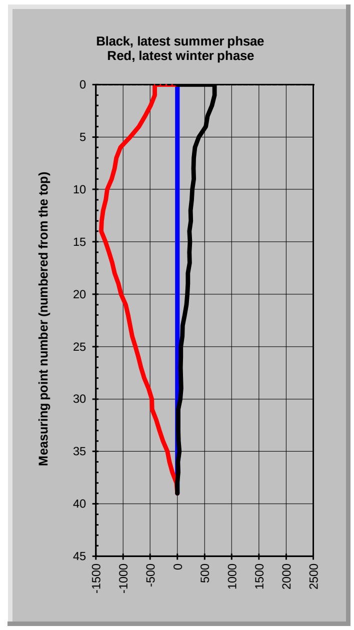
Legend: 01/99 03/00 03/01 02/02 02/05 12/10 - 02/12 02/13 03/14

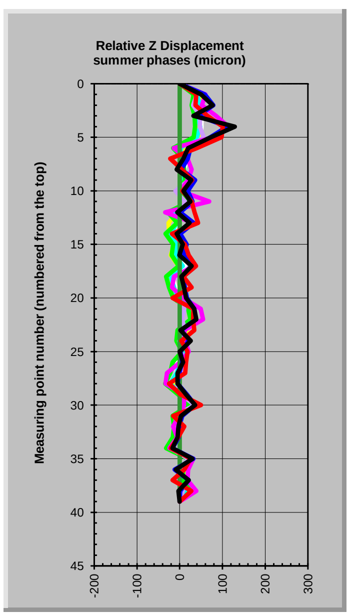
Rock/concrete contack

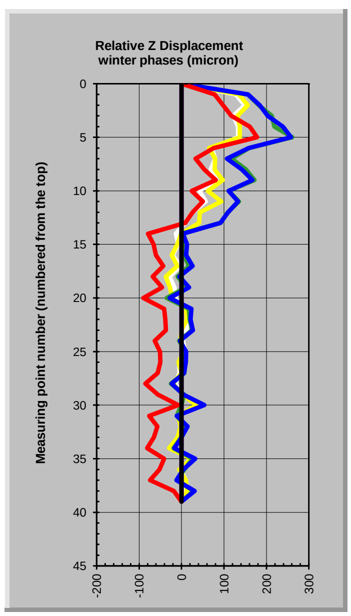
Base date

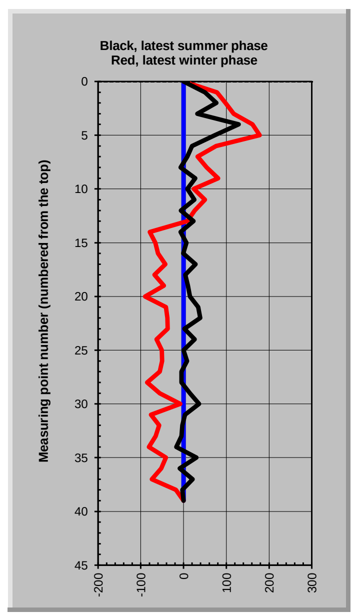






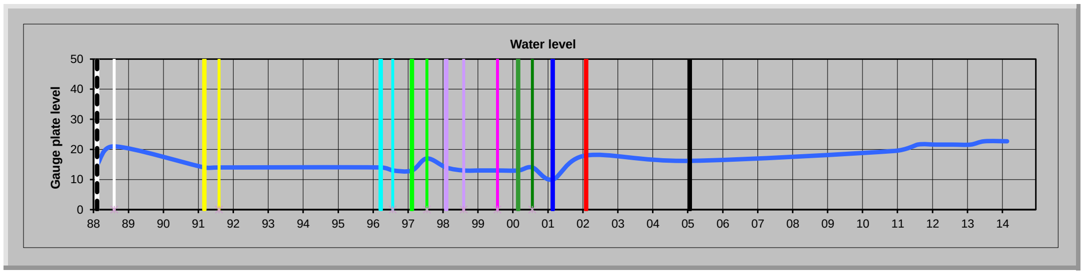






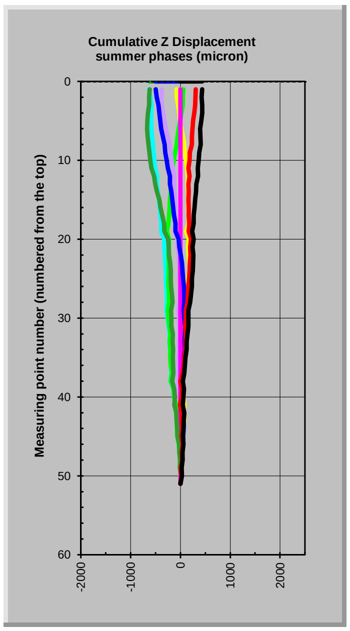
Positive is shortening

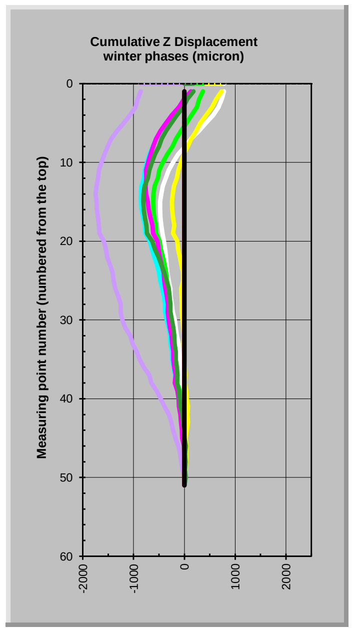
Vrvm2 1988 - 2005

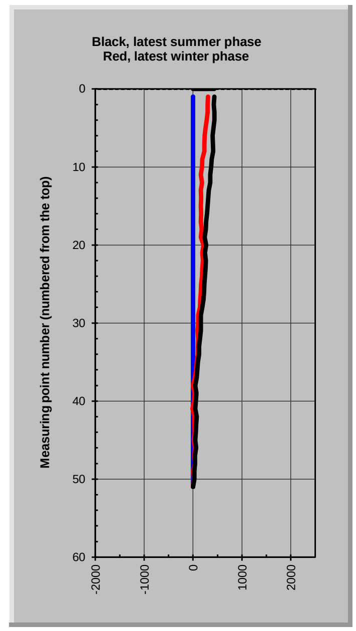


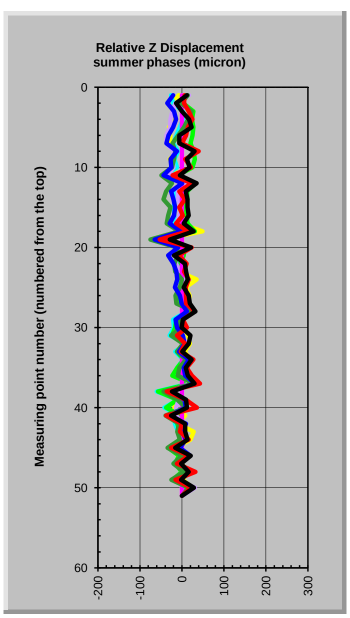
Legend: 02/88 03/91 03/96 02/97 02/98 - 03/00 03/01 02/02 02/05

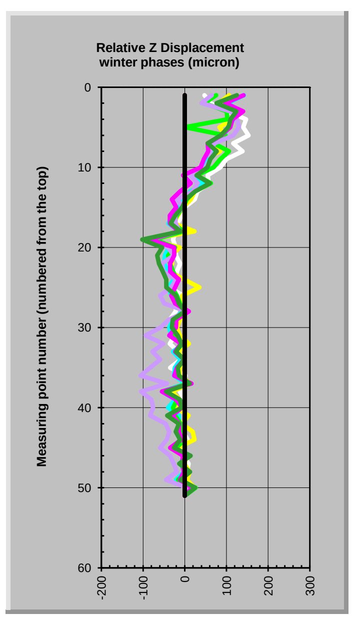
Rock/concrete contack Base date

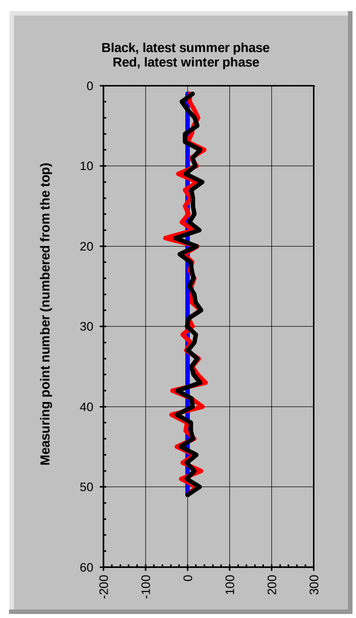






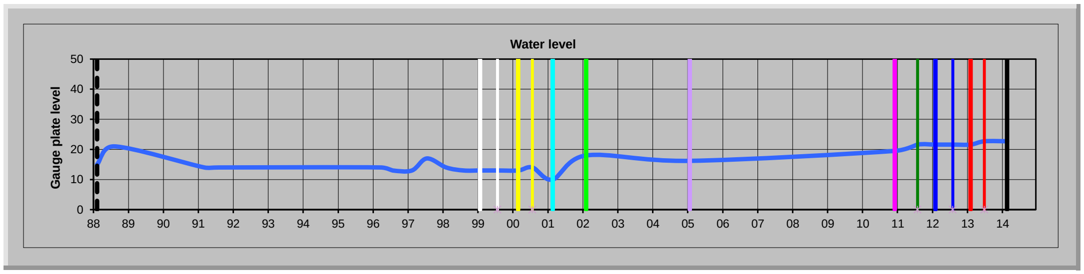






Positive is shortening

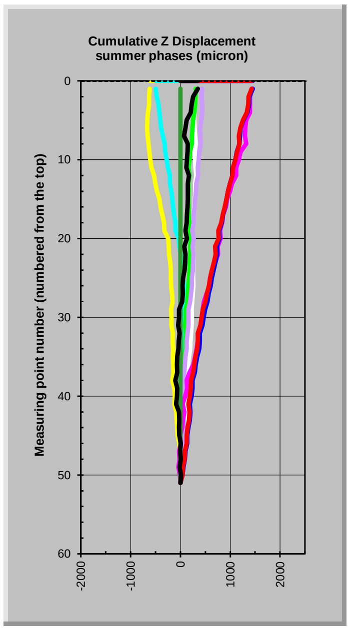
Vrvm2 1999 - 2014

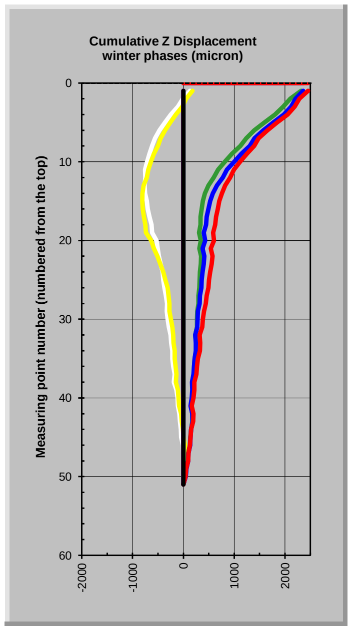


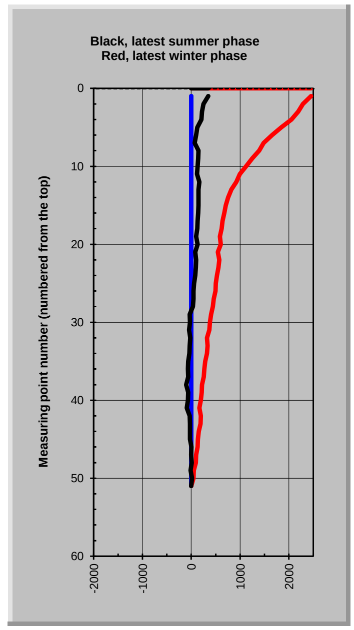
Legend:

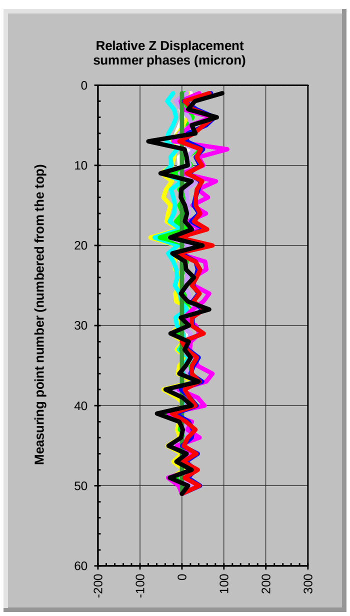
01/99 03/00 03/01 02/02 02/05 12/10 - 02/12 02/13 03/14

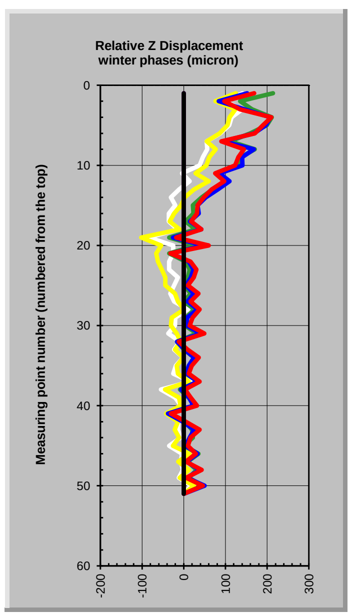
Rock/concrete contack Base date

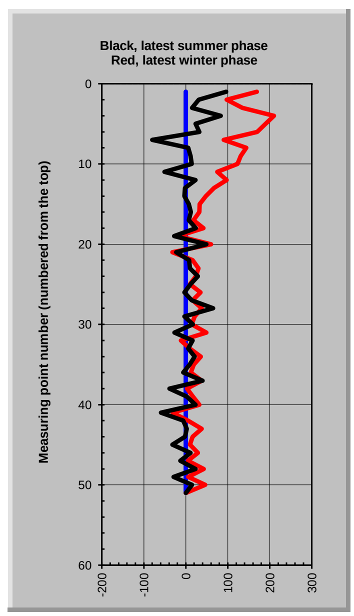












Positive is shortening

APPENDIX D: RESULTS OF THE FEM ANALYSIS OF NQWEBA DAM

2D Static Analyses Results for the Central Section (Section 1)

Load Case and Analysis Type	Displacement (mm)	Stress at heel (MPa)	Stress at toe (MPa)
No Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.043	0.059	-1.020
"Non-linear" (Drucker Prager) Analysis	2.044	0.060	-1.020
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	4.359	0.569	-1.559
"Non-linear" (Drucker Prager) Analysis	4.429	0.424	-1.575
50% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.050	0.208	-0.955
"Non-linear" (Drucker Prager) Analysis	2.022	0.214	-0.938
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	4.365	0.682	-1.494
"Non-linear" (Drucker Prager) Analysis	4.461	0.440	-1.506
100% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.125	0.341	-0.933
"Non-linear" (Drucker Prager) Analysis	2.008	0.345	-0.858
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	4.441	0.815	-1.472
"Non-linear" (Drucker Prager) Analysis	4.502	0.506	-1.448

Note: Positive stress is tensile

2D Response Spectrum Analysis Results for the Central Section (Section 1)

Response Spectrum Analysis		
Load Case	**Stress at heel (MPa)**	**Stress at toe (MPa)**
Abnormal Seismic Load	S_{22} (Heel)	S_{22} (Toe)
OBE	0.121	-6.642e-4
Extreme Seismic Load	S_{22} (Heel)	S_{22} (Toe)
MCE	0.200	-1.103e-3

2D Static Analyses Results for the Left Section (Section 2)

Load Case and Analysis Type	Displacement (mm)	Stress at heel (MPa)	Stress at toe (MPa)
No Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.707	0.100	-0.807
"Non-linear" (Drucker Prager) Analysis	1.707	0.100	-0.807
Extreme Load (SEF)	Downstream	S22	S22
Linear Elastic Analysis	3.897	0.733	-1.545
"Non-linear" (Drucker Prager) Analysis	4.084	0.390	-1.596
50% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.754	0.222	-0.776
"Non-linear" (Drucker Prager) Analysis	1.756	0.217	-0.776
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	3.945	0.807	-1.515
"Non-linear" (Drucker Prager) Analysis	4.180	0.419	-1.578
100% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.816	0.341	-0.756
"Non-linear" (Drucker Prager) Analysis	1.824	0.312	-0.758
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	4.006	0.925	-1.494
"Non-linear" (Drucker Prager) Analysis	4.289	0.475	-1.571

Note: Positive stress is tensile

2D Response Spectrum Analysis Results for the Left Section (Section 2)

Response Spectrum Analysis		
Load Case	**Stress at heel (MPa)**	**Stress at toe (MPa)**
Abnormal Seismic Load	S_{22} (Heel)	S_{22} (Toe)
OBE	0.108	-6.087e-4
Extreme Seismic Load	S_{22} (Heel)	S_{22} (Toe)
MCE	0.179	-1.010e-3

2D Static Analyses Results for the Right Section (Section 3)

Load Case and Analysis Type	Displacement (mm)	Stress at heel (MPa)	Stress at toe (MPa)
No Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.069	0.029	-0.748
"Non-linear" (Drucker Prager) Analysis	1.069	0.029	-0.748
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.756	0.764	-1.306
"Non-linear" (Drucker Prager) Analysis	2.886	0.400	-1.351
50% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.087	0.131	-0.707
"Non-linear" (Drucker Prager) Analysis	1.087	0.131	-0.707
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.774	0.840	-1.265
"Non-linear" (Drucker Prager) Analysis	2.940	0.403	-1.321
100% Uplift			
Service Load (FSL)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	1.129	0.226	-0.689
"Non-linear" (Drucker Prager) Analysis	1.129	0.223	-0.689
Extreme Load (SEF)	Downstream	S_{22} (Heel)	S_{22} (Toe)
Linear Elastic Analysis	2.816	0.909	-1.247
"Non-linear" (Drucker Prager) Analysis	3.013	0.428	-1.315

Note: Positive stress is tensile

2D Response Spectrum Analysis Results for the Right Section (Section 3)

Response Spectrum Analysis		
Load Case	**Stress at heel (MPa)**	**Stress at toe (MPa)**
Abnormal Seismic Load	S_{22} (Heel)	S_{22} (Toe)
OBE	0.0868	-5.085e-4
Extreme Seismic Load	S_{22} (Heel)	S_{22} (Toe)
MCE	0.144	-8.416e-4

3D Static Analyses Results

Load Case and Analysis Type	Displacement (mm)	Maximum Principal Stress (MPa)	Minimum Principal Stress (MPa)
No Uplift			
Service Load (FSL)	Downstream	S_1	S_3
Linear Elastic Analysis	2.269	0.336/0.527*	-1.226
"Non-linear" (Drucker Prager) Analysis	2.269	0.336/0.527*	-1.226
Abnormal Load (SEF)	Downstream	S_1	S_3
Linear Elastic Analysis	4.659	0.852	-1.986
"Non-linear" (Drucker Prager) Analysis	4.688	0.575/0.653*	-2.001
50% Uplift			
Service Load (FSL)	Downstream	S_1	S_3
Linear Elastic Analysis	2.277	0.456/0.584*	-1.155
"Non-linear" (Drucker Prager) Analysis	2.277	0.456/0.584*	-1.155
Abnormal Load (SEF)	Downstream	S_1	S_3
Linear Elastic Analysis	4.666	0.995	-1.914
"Non-linear" (Drucker Prager) Analysis	4.732	0.667/0.694*	-1.942
100% Uplift			
Service Load (FSL)	Downstream	S_1	S_3
Linear Elastic Analysis	2.349	0.579/0.644*	-1.120
"Non-linear" (Drucker Prager) Analysis	2.349	0.578/0.644*	-1.120
Abnormal Load (SEF)	Downstream	S_1	S_3
Linear Elastic Analysis	4.739	1.133	-1.878
"Non-linear" (Drucker Prager) Analysis	4.844	0.763	-1.919

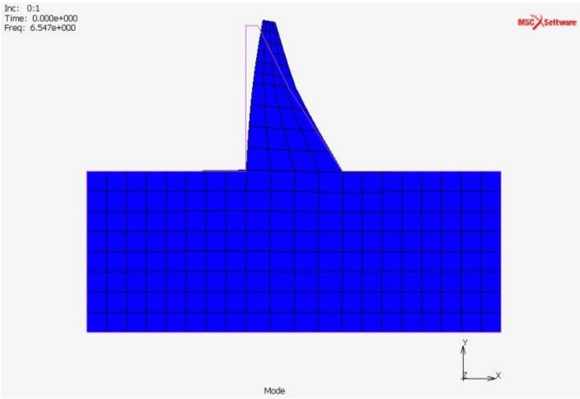
Note: Positive stress is tensile
The asterisk (*) refers to the maximum principal stresses occurring in the foundation

3D Response Spectrum Analysis Results

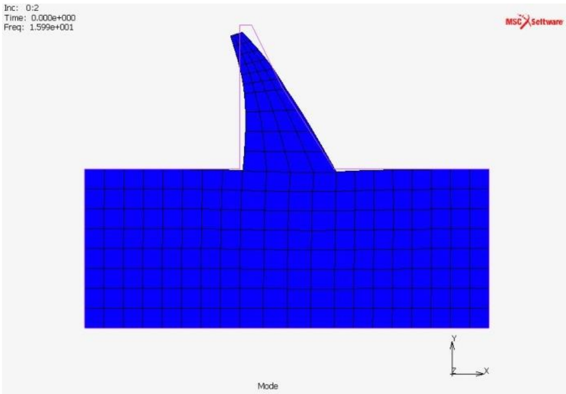
Response Spectrum Analysis		
Load Case	**Maximum Principal Stress (MPa)**	**Minimum Principal Stress (MPa)**
Abnormal Seismic Load	S_1	S_3
OBE	0.111	-0.0130
Extreme Seismic Load	S_1	S_3
MCE	0.184	-0.0216

APPENDIX E: FIRST SIX MODE SHAPES OF THE DYNAMIC MODAL FEM ANALYSIS OF NQWEBA DAM

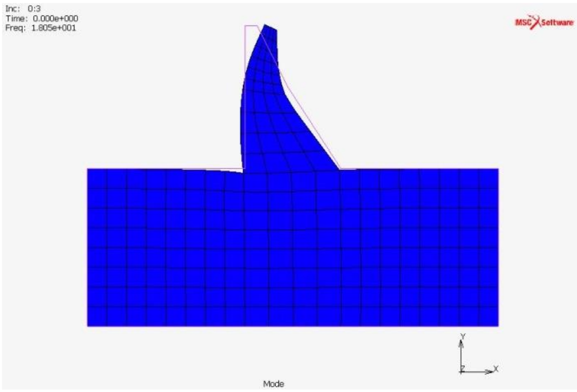
2D Mode Shapes and Natural Frequencies of the Central "River" Section



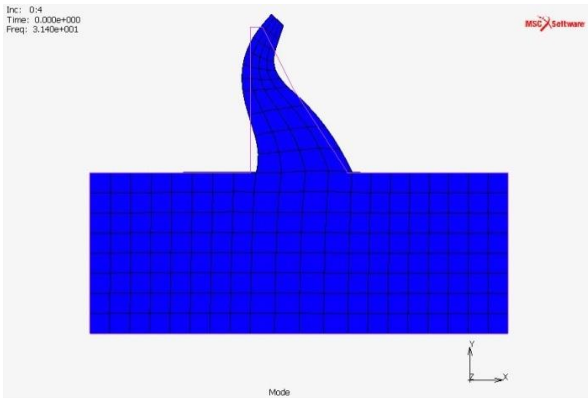
Mode 1: 6.55 Hz



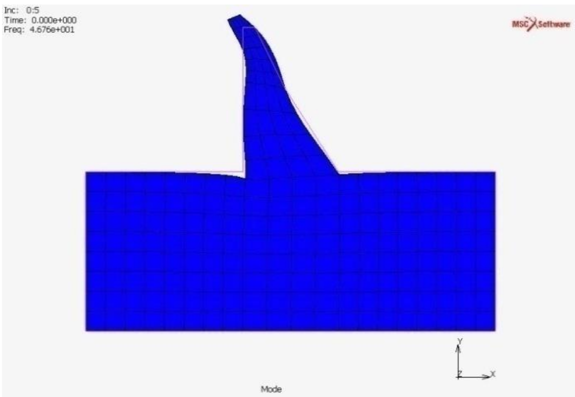
Mode 2: 15.99 Hz



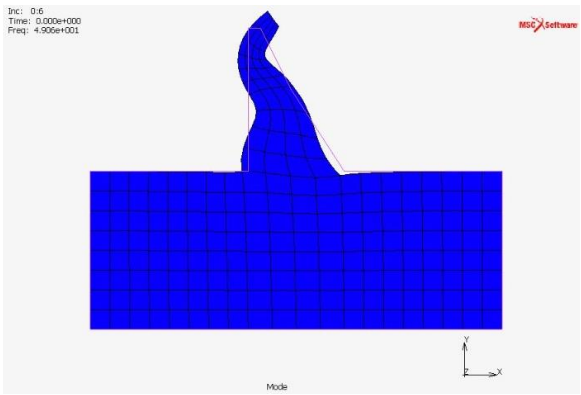
Mode 3: 18.05 Hz



Mode 4: 31.40 Hz

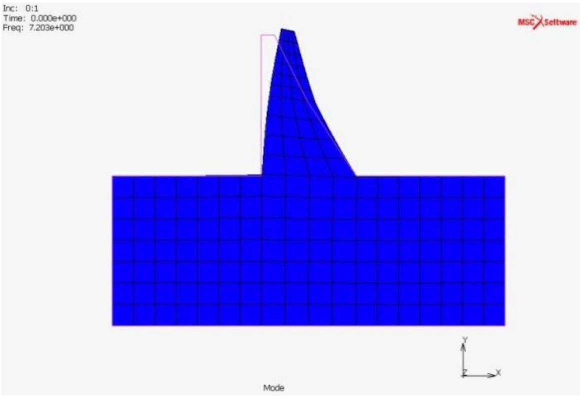


Mode 5: 46.76 Hz

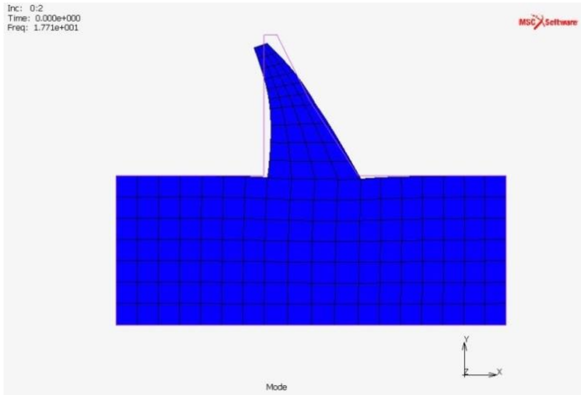


Mode 6: 49.06 Hz

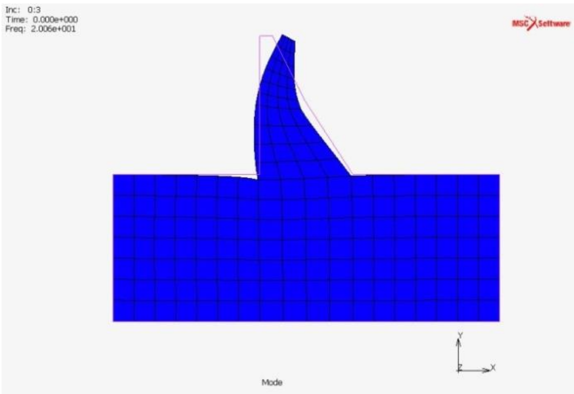
2D Mode Shapes and Natural Frequencies of Left Section



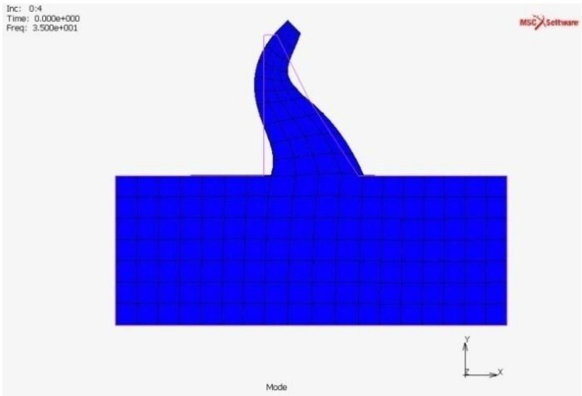
Mode 1: 7.20 Hz



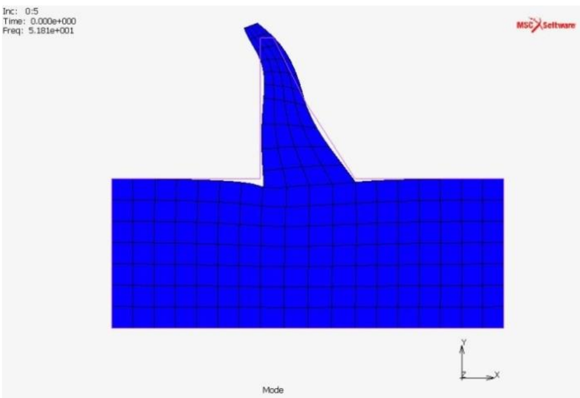
Mode 2: 17.71 Hz



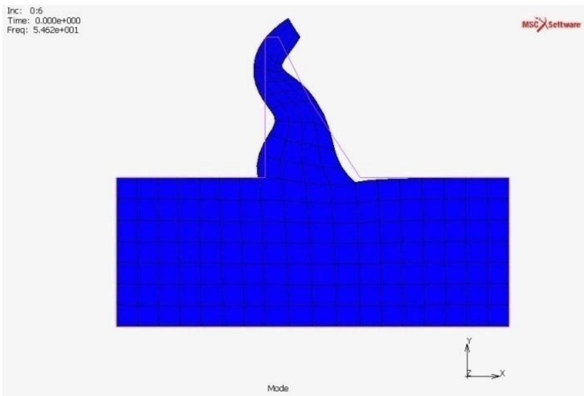
Mode 3: 20.06 Hz



Mode 4: 35.00 Hz

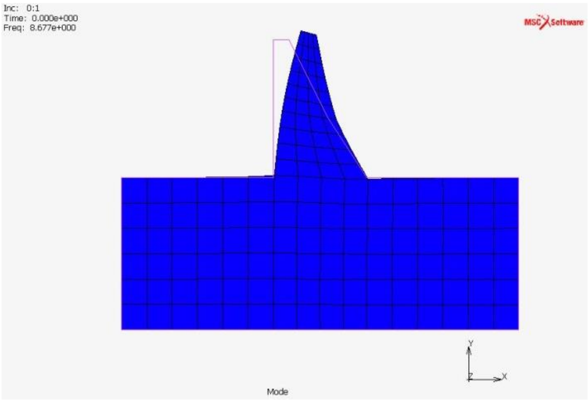


Mode 5: 51.81 Hz

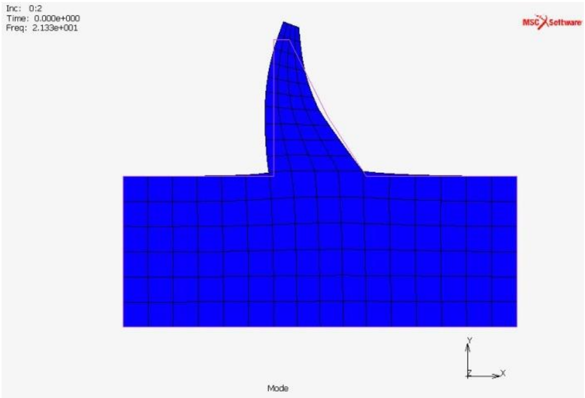


Mode 6: 54.62 Hz

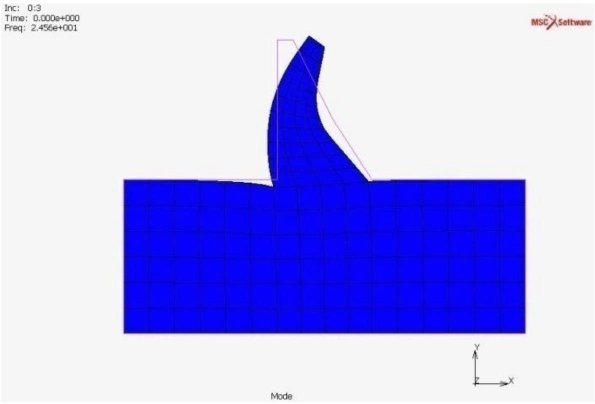
2D Mode Shapes and Natural Frequencies for the Right Section



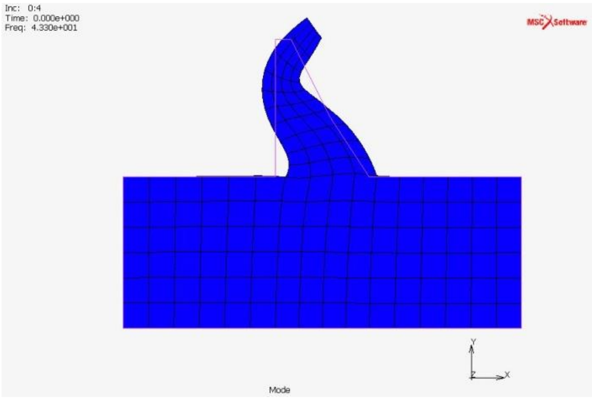
Mode 1: 8.68 Hz



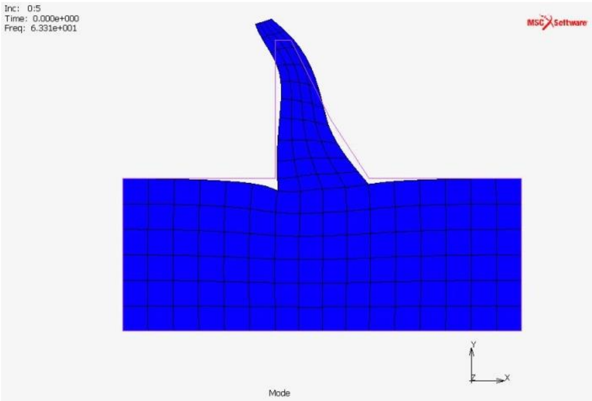
Mode 2: 21.33 Hz



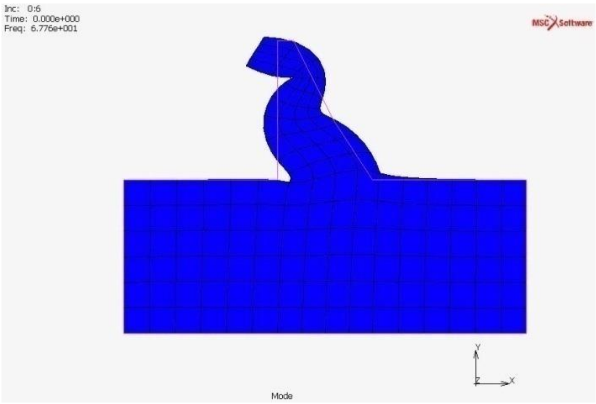
Mode 3: 24.56 Hz



Mode 4: 43.30 Hz

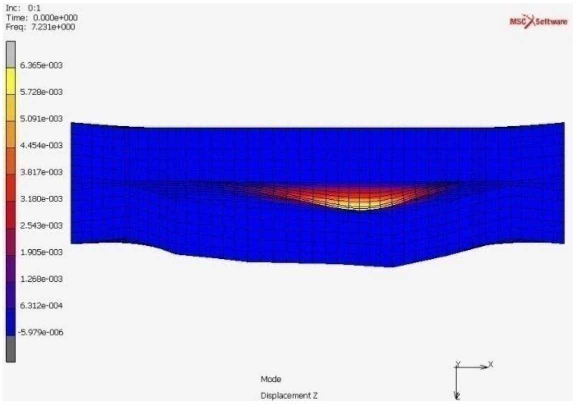


Mode 5: 63.31 Hz

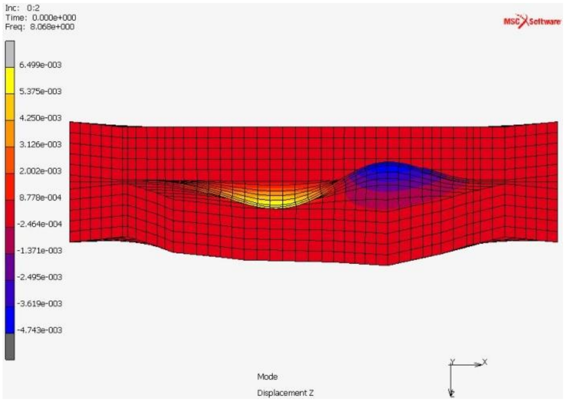


Mode 6: 67.76 Hz

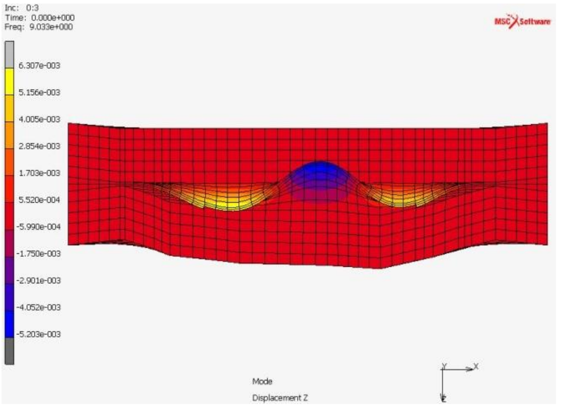
3D Mode Shapes and Natural Frequencies



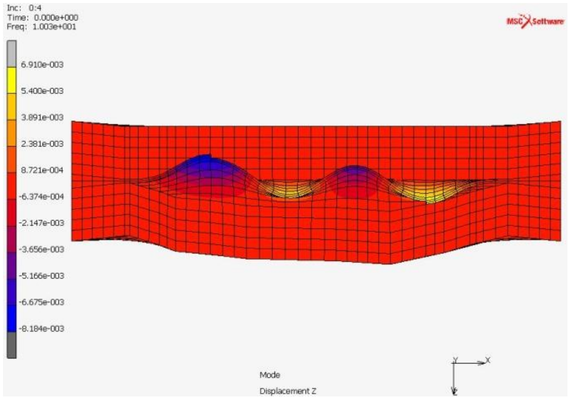
Mode 1: 7.23 Hz



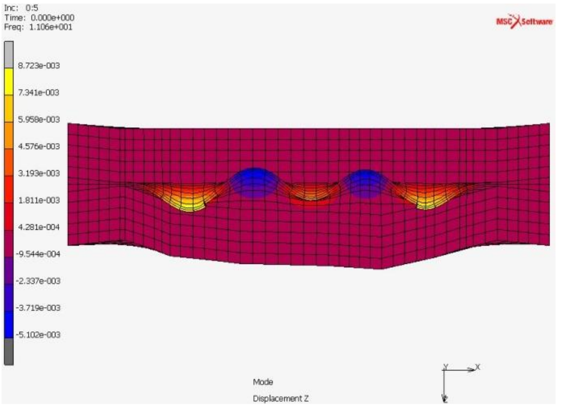
Mode 2: 8.07 Hz



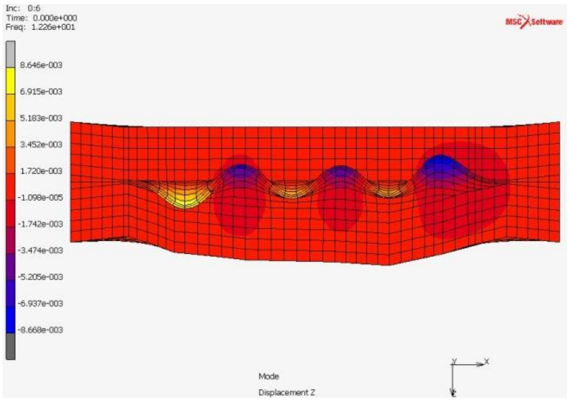
Mode 3: 9.03 Hz



Mode 4: 10.03 Hz



Mode 5: 11.06 Hz



Mode 6: 12.26 Hz